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Standardization of civil engineering works of small high-head hydropower plants and development of an optimization tool

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PREFACE

Small hydropower plants with an installed capacity below 10 MW are a renewable source of energy which can still be strongly developed all over the world. Nevertheless for the development of small hydro, simple and generally applicable procedures and methods for the design are lacking, which could limit the engineering costs. The study carried out in the framework of the European Project FP5 “Thematic Network on Small Hydropower” had the goal to provide a general guide with regard to economical design and practical realization of the main structures of small hydropower plants and their interaction.

Based on examples of best practice and theoretical background, Mohammadreza Andaroodi established a standardized design of every structure of a high head small hydropower plant. This allows a fast evaluation and comparison of different alternatives in the feasibility phase of the project. Mr. Andaroodi estimated the standardized civil works in terms of concrete volume, reinforcement, formwork, excavation and backfilling. Very useful relationships between the above items and the installed discharge were obtained. Based on local unit prices, cost functions of the structures can be defined and implemented in an optimization program called POPEHYE. A first version of this software was developed by G. Gatti under the guidance of Dr Jean-Louis Boillat. Mr. Andadaroodi completed the software with the standardized civil works cost functions. The optimization tool allows a step by step design and economical optimization of the layout, head and installed discharge of small hydro in a very efficient way without neglecting the requirements of best design practice.

The study was performed as a thesis of the master of advanced studies (MAS) in hydraulic schemes at LCH-EPFL and supported by the European Project FP5 “Thematic Network on Small Hydropower” (see www.esha.be).

Prof. Dr. Anton J. Schleiss

With affection and love, to

My wife ***Homa***

ABSTRACT

Small hydropower has been identified as one of the important energy sources that can provide convenient and uninterrupted energy to remote rural communities or industries. This type of scheme is recognized as a renewable source of energy, which is economic, non-polluting, environmentally sustainable and ideal for rural electrification. Hydropower is typically defined as “small” for an installed capacity less than 10 MW. Small hydro deserves to have its development accelerated in most parts of the world and developing countries.

For development of such small plants, simple and generally applicable procedures and methods for the design are lacking. The interest of small hydropower resources is increasing but few published guidelines exist for project design. This study aims at providing a general guidance with regard to economical design and practical realization of the main components of small plants and their interactions.

A high-head small hydropower plant contains the following basic components: water intake, settling basin, headrace canal or pipe, forebay or surge tank and penstock. Based on examples of best practice and theoretical background, a standardized design of each component has been established. Standardization of civil works covers typical design charts of these various components which contribute to a fast evaluation and comparison of different alternatives in the pre-feasibility stage of a project. Following the estimation of all civil works in terms of concrete volume, reinforcement, formwork, excavation and backfilling, the designer will obtain the cost functions according to the local unit prices, which will be used later on in the optimization process and detailed design.

After standardization of all civil works of a small hydro, the final volumetric curves and cost functions are implemented in an optimization program called “POPEHYE”. This software was developed jointly at the Laboratory of Hydraulic Constructions of the Ecole Polytechnique Fédérale de Lausanne and at Yverdon University of Applied Sciences (Switzerland). It allows a step by step design and optimization for evaluation of different alternatives of small hydropower plants, according to layout, head and discharge.

By implementing the standardized design of the main components, the program’s accuracy related to optimization strategy was increased.

The final result of the economical analysis of a project is the graphical presentation of the production cost, net benefit and economic efficiency as a function of design discharge of the small hydropower plant. The designer is then able to select an optimum design discharge for the project through these economic parameters. Having the maximum benefit is thereby considered as the most important factor. This optimization of design discharge or installed capacity is performed with the assumed total head, considering all head losses in the waterway systems.

The program “POPEHYE” completes the preliminary design of the main structures for the optimum design discharge. Detailed design of all components is accomplished during standardization procedures which determine the geometry and dimension of structures by using standard drawings and equations.

This standardization and optimization represents a practical guideline for better realization and implementation of a small hydropower project.

RESUME

La petite hydraulique permet la production d'électricité de ruban de manière efficace pour l'approvisionnement énergétique de villages isolés ou d'industries locales. Il s'agit d'une source d'énergie renouvelable, non polluante et économique. Elle est idéale pour l'électrification du milieu rural. Le terme "petite hydraulique" s'applique aux installations ayant une capacité de production inférieure à 10 MW. Le potentiel d'expansion d'une telle source d'énergie est important, en particulier dans les pays en voie de développement.

L'expansion de ce moyen de production est cependant encore limité par le manque de méthodes de planification et de dimensionnement efficaces. En effet, malgré l'intérêt grandissant pour de telles installation, peu de recommandations pratiques sont actuellement disponibles. L'objectif de cette étude est donc de fournir une méthode de planification et de dimensionnement de mini-centrales hydrauliques pour l'ingénieur projeteur en considérant les aspects techniques et économiques des différents composants de ces ouvrages.

Un aménagement de production hydroélectrique de type mini-centrale se compose en général des ouvrages suivants: prise d'eau, dessableur, canal ou conduite d'amenée, chambre de mise en charge et conduite en charge. Une procédure de choix et de dimensionnement de ces ouvrages est proposée sur la base d'examples partiques et des théories existantes. Une standardisation des différents ouvrages a été effectuée, qui permet une évaluation rapide et précise d'un projet au stade de l'étude de faisabilité. En se basant sur un métré considérant les volumes de béton, d'ouvrages de renforcement, des coffrages, de l'excavation, du soutènement et du remblai, une évaluation chiffrée du coût des travaux est proposée.

Après cette phase de standardisation des ouvrages hydrauliques, les courbes volumétriques et fonctions de coût sont implémentées dans le logiciel "POPEHYE". Ce programme a été développé conjointement au Laboratoire de Constructions Hydrauliques de l'Ecole Polytechnique Fédérale de Lausanne et à l'Ecole d'Ingénieurs d'Yverdon, en Suisse. Il permet la conception et le dimensionnement pas par pas de différentes variantes de projets de mini-centrales hydroélectriques en fonction de la configuration du terrain, de la charge et du débit. Grâce à la standardisation des différents composants de génie civil, le programe permet enfin l'optimisation des variables de projet telles que le débit équipé.

Le résultat final de l'analyse économique du projet est une représentation graphique des coûts de production, du bénéfice net et de la rentabilité en fonction du débit équipé. Le projeteur est ainsi capable de déterminer le débit équipé optimal en fonction de l'efficacité économique de son projet. L'analyse prend en compte les aspects hydrauliques tels que les pertes de charge.

Le programme "POPEHYE" permet donc l'analyse préliminaire d'aménagements hydroélectriques de type mini-centrales et la détermination des principales variables de projet. Le détail des ouvrages hydrauliques est déterminé grâce à la standardisation des différents composants en fonction de géométries types.

Ce nouvel outil permettra ainsi d'aboutir rapidement à un avant-projet de mini-centrale réaliste.

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List of symbols

P	Real power (kW)
ρ_w	Density of water (kg/m^3)
g	Gravity acceleration (m/s^2)
Q	Design discharge (m^3/s)
η	Global efficiency (%)
H	Gross head (m)
E	Annual energy production (kWh/yr)
Vol	Yearly mean volume of turbined water (m^3/yr)
c	Trashrack coefficient
B	Width of structure (m)
L	Length of structure (m)
β	Slope angle of trashrack (^{o})
K_c	Correction factor in intake
h_{cr}	Critical depth (m)
a	Opening between adjacent bars in intake (mm)
b	Center spacing of bars in intake (mm)
V_D	Settling velocity in flowing water (m/s)
V_T	Mean flow velocity in settling basin (m)
V_{cr}	Critical flow velocity in settling basin (m/s)
h	Water depth (m)
ρ_s	Density of sediment (kg/m^3)
d	Design grain size in settling basin (mm)
K	Strickler coefficient (m$^{1/3}$/s)
V_{D0}	Settling velocity in still water (m/s)
α	Reduction factor (1/m$^{1/2}$)
A	Flow area (m^2)
V	Water velocity (m/s)
S	Canal bed slope
R	Hydraulic radius (m)
θ	Angle of canal banks above horizontal (^{o})
ϕ	Central angle for free flow in circular section (^{o})
D	Diameter of circular conduit (m)
h_t	Submergence depth (m)
Q_s	Spillway discharge (m^3/s)
C_d	Discharge coefficient of spillway
B_s	Spillway width (m)
h_s	Spillway head (m)
T_c	Critical time in penstock surge (s)
a_c	Wave flow velocity (m/s)
E_T	Module of elasticity of penstock (N/m^2)
E_w	Module of elasticity of water (N/m^2)
t	Wall thickness of penstock (m)
ΔH	Head loss (m)
f	Friction coefficient in Darcy-Weisbach equation
k_s	Average roughness height on the pipe (m)
Re	Reynolds number
ν	Kinematic viscosity of water (m^2/s)
Z_{max}	Maximum oscillation in surge tank (m)
Z_{min}	Minimum oscillation in surge tank (m)

A_0	Area of pressurized headrace pipe (m^2)
L'	Length of headrace pipe (m)
A_s	Area of surge tank (m^2)
v^*	Reference velocity (m/s)
P_{max}	Total maximum pressure in penstock (N/m^2)
σ_t	Admissible stress of steel (N/m^2)
W_s	Total steel weight (kg)
ρ_t	Density of steel (kg/m^3)
D'	External diameter of pipe (m)
a_n	Annuity factor
i	Rate of interest (%)
n	Amortization time (yr)
C_{an}	Annual cost (unit cost/year)
C_T	Total cost ((unit cost/year)
R	Resulting force on anchor block (N)
R_x	Horizontal components of force on anchor block (N)
R_y	Vertical components of force on anchor block (N)
α_a	Angle of d/s part of penstock with horizontal in anchor point (^{o})
β_a	Angle of u/s part of penstock with horizontal in anchor point (^{o})
φ	Angle of total force on anchor with horizontal (^{o})
FL_{effx}	Contributive effective surface (km^2)
L_{Ge}	Cumulative length of river branches in the watershed (km)
FL	Surface area of watershed (km^2)
f_x	Infiltration loss (mm/h)
Vo_x	Wet volume (mm)
TR_x	Precipitation time (h)
$T1_x$	Wet duration (h)
$T2_x$	Duration of total flow (h)
R_x	Precipitation intensity of duration TR_x (mm/h)
HQ_x	The flood discharge (m^3/s)
r_s	Contribution of snow melting (mm/h)
FL_b	Impermeable surface (km^2)
Q_{GLe}	Original discharge of glacier (m^3/s)
GLE_{AN}	Proportion of glacier surface of watershed
K_{Gang}	Coefficient in Koella formula
U	Reduced variable of Gumbel
a_g	Mode of distribution in Gumbel
T	Return period (yr)
S	Standard deviation of discharge values in Gumbel
$\overline{Q}$	Average value of maximum annual discharges (m^3/s)
m	Number of each row of discharge in Gumbel
n_g	Total number of discharge values in Gumbel
P	Probability function related to return period in Gumbel
u_m	Reduced variable for each row of discharges in Gumbel
q_{max}	Maximum specific discharge of flood (m^3/s per km^2)
α'	Parameter in Hofbauer empirical equation
φ_0	Coefficient in Melli empirical equation
ψ_0	Coefficient in Müller empirical equation
α_L	Coefficient in Lauterburg empirical equation
W	River width (m)
P_{prod}	Resale price or production cost (unit cost/yr)

Be	Net benefit (unit cost/yr)
η_e	Economical efficiency (%)
C_m	Maintenance cost (unit cost/yr)

Glossary

EU	European Union
SHP	Small Hydropower Plant
ESHA	European Small Hydropower Association
TNSHP	Thematic Network on Small Hydropower Plant
FDC	Flow Duration Curve
IDF	Intensity – Duration - Frequency
PVC	polyvinyl chloride
POPEHYE	Predimensionnement et optimisation économique des petits aménagements hydroélectriques
LCH	Laboratory of Hydraulic Constructions
EPFL	Ecole Polytechnique Fédérale de Lausanne
EIVD	Ecole d'Ingénieurs d'Yverdon

1 Introduction

Small Hydropower Plants (SHP) have been identified as one of the most important energy sources that can provide convenient and uninterrupted energy to remote rural communities or industries. This hydropower is recognized as a renewable source of energy, which is economic, non-polluting, environmentally sustainable and ideal for rural electrification.

1.1 Definition of SHP

There is no general consensus in European Union (EU) on the definition of small hydropower. Some countries like Portugal, Spain, Ireland, and now, Greece and Belgium, accept 10 MW as the upper limit for installed capacity. In Italy the limit is fixed to 3 MW; in France the limit was established to 8 MW and UK favor 5 MW. The European Small Hydropower Association has finally stated the following definition referred to the installed capacity at the plant [12]:

- Micro hydro plants: up to 100 kW
- Mini hydro plants: up to 500 kW
- Small hydro plants: up to 10 MW

In this study, small hydro schemes are typically defined as having an installed capacity of less than 10 MW.

1.2 SHP development

Small hydro deserves to have its development accelerated in most part of the world and developing countries. Nearly 22% of the world's electricity production comes from hydropower installations, many of which are small hydropower plants.

Small hydropower accounts for approximately 7% of total hydro generation in Europe [21]. The present capacity for 30 European countries is shown in Figure 1.1. The total installed SHP capacity stands at 12600 MW and production is estimated at 50000 GWh. Leading countries are Italy, France, Germany, Spain, Sweden, Norway, Austria and Switzerland which combine 86% of SHP capacity and production in Europe.

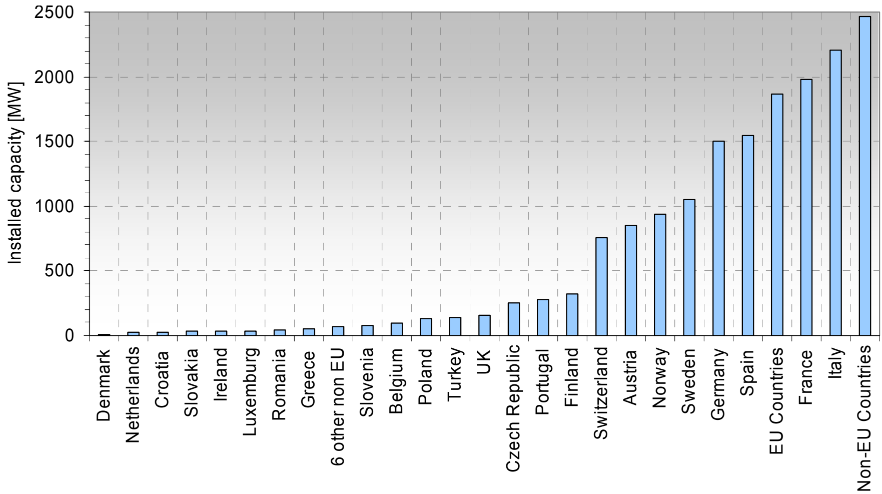


Figure 1.1: Installed capacity and production of SHP (up to 10 MW) in 30 European countries [21]

There are more than 17400 SHP installed in Europe and the number of such plants in Switzerland is about 1100, with total installed capacity of about 750 MW.

In Switzerland, SHP was the most common way of electricity generation. In the early 20^{th} century, the water rights registration office recorded nearly 7000 SHP (<10 MW) of which more than 90% were plants below 300 kW consisting of water wheels and micro turbine [33]. Until 1985 approximately 1000 SHP survived in Switzerland, of nearly 700 with a capacity below 300 kW.

Renewed interest in the technology of small scale hydropower started in China. Estimations say that between 1970 and 1985 nearly 76000 small scale stations have been built there.

1.3 History of SHP

Water power has contributed to the development of mankind since Biblical times. Indications of the use of waterwheels for milling, pumping, and other functions date back to 300 B.C. in Greece, although they were probably in use before this time [25].

SHP history is characterized by two main periods, based on the use of water energy:

- First period: when hydraulic energy is meant to generate mechanical energy
- Second period: when hydraulic energy is meant to generate electrical energy

The first hydraulic machines appeared about 2200 years ago probably almost at the same time in China as in the Mediterranean basin. In Europe, before the general electric networks extensions, about ten thousands of small hydraulic installations existed and were used in sawmills tanneries, pulp and paper mills, mechanical shops, etc. Due to the electricity development, hydraulic turbines had to be adapted to this new energy use. It should be emphasized that current turbines have nothing to do with old water wheels. They have been improved to reach high performances and efficiency, while simplified geometry and technical solutions assuring a high reliability.

1.4 Advantages of SHP

SHP contributes towards sustainable development by being economically feasible, respecting the environment and allowing decentralized production for the development of scattered population. Furthermore some other advantages can be underlined regarding the following fields [25]:

- SHP is the most important energy source in what concerns no carbon dioxide or any other air emissions and no solid or liquid waste production.
- SHP is a non-consumptive generator of electrical energy, utilizing a renewable resource which becomes continuously available through the hydrologic cycle by the energy of the sun.
- SHP is essentially non-polluting and releases no heat. Adverse environmental impacts are negligible and, for small installations, may be totally eliminated.
- SHP can be designed and built within one or two year's time. Licensing requirements are minimal, equipments are readily available, and construction procedure is well known.
- SHP requires some types of water control, inclining full regulation of watershed discharge. They are thus an important element in the multipurpose utilization of water resources and can reduce flood damage.
- In remote areas using relatively simple technology can be a catalyst in mobilizing productive resources and creating enhanced economic opportunities for local residents.
- SHP is a reliable resource within the hydrologic limitations of the site. The relative simplicity of hydraulic machinery makes energy instantly available as needed. Since no heat is produced, equipments have a long life and malfunctions are rare.

- SHP is characterized by reliability and flexibility of operation, including fast start-up and shut-down in response to rapid change in demand. It thus becomes a valuable part of any large electrical system, increasing overall economy, efficiency, and reliability.
- SHP requires few operating personnel. Some small-scale installations are operated entirely by remote control. Independency on fuel, together with equipment's long life, makes hydroelectric power installations resistant to inflation.
- SHP developing can make maximum use of local materials and labor. Compared to thermal facilities, small hydro usually provides more local employment in civil works.
- SHP economic feasibility is improving compared to other energy sources that use finite fuels.
- Current SHP technology assists in refurbishing old SHP.
- Since SHP is located close to the consumers, transmission losses can be reduced and the electricity supply lines are eased.

1.5 Organization on SHP

There are a lot of organizations that are involved in SHP research activities and developments. The most important in Europe is the European Small Hydropower Association (ESHA), which is a non-profit international association representing the sector of small hydropower. The association was founded in 1989 as an initiative of the European Commission. In the framework of the 5th European research program, the Thematic Network on Small Hydropower (TNSHP) has been started in 2003. It aims to identify future research and market needs of the SHP sector within the EU and the candidate countries in order to overcome barriers and promote a better exploitation of the resource as regards costs, public acceptance, integration into energy systems, technological issues, environmental impacts and the fulfillment on installed capacity.

1.6 Types of SHP

Various possibilities exist for the general lay-out of a hydro scheme, depending on the local situation. Based on the head, schemes can be classified in three main categories [8].

- High-head SHP: 100 m head and above (focus of present study)
- Medium head SHP: 30 to 100 m head.
- Low head SHP: 2 to 30 m head

These ranges are not rigid but are merely means of categorizing sites. Higher heads require less water to produce a given amount of power.

The large majority of small hydro plants are run-of-river schemes, meaning simply that the turbine generates when the water is available in the river and there is no storage capacity. When the river dries up and the flow falls below some predetermined discharge of scheme, the power generation ceases.

1.6.1 High-head SHP

In this type of small hydropower plant, the water is diverted through an intake in the river bank or bed. A settling basin is placed after the intake structure to remove sand particles from the flowing water. Then a headrace canal follows the contour of the hillside to provide the required head for energy production. After that the water enters a forebay and passes into a closed pipe known as a penstock. This last structure is connected at a lower elevation to a turbine located in the power house. At the outlet of the turbine, the water is discharged to the river, via the tailrace. Figure 1.2 shows the main components of a high-head small hydropower plant.

1. Water intake
2. Settling basin
3. Headrace canal
4. Forebay
5. Penstock
6. Power house

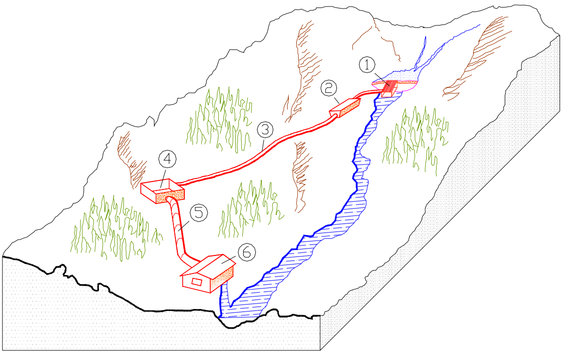


Figure 1.2: Main component of a high-head small hydropower plant

1.6.2 Low head SHP

These types of schemes are typically built in wide and flat river valley. Two technological options can be selected. Either the water is diverted to a power intake with a short penstock, or the head is created by a small dam equipped with sector gates, an integrated intake and power house (Figure 1.3).

Figure 1.3: Examples of a low head SHP

1.7 Civil works of SHP

Civil engineering works of a normal high-head small hydropower plant contain the following basic components: <u>intake</u>, <u>settling basin</u>, <u>headrace canal or pipe</u>, <u>forebay or surge tank</u> and <u>penstock</u>. Each of these components serves specific purposes (Figure 1.4):

- **Water intake:** an intake permits a controlled flow of water from a river or stream into a conduit, which eventually conveys it to the power plant. The intake is a component of virtually every hydropower scheme and its proper design is essential for trouble-free operation of the civil works. One of the major functions of the intake is to minimize the entrance of floating debris and sediments carried by the incoming water.

- **Settling basin (sand trap):** this structure is normally located after the intake and removes the sediment particles from the flow in order to avoid its entrance to the headrace system and turbine.
- **Headrace canal:** The term headrace canal signifies the component of a SHP scheme used to convey water in a relatively large distance from the stream to the inlet of the penstock. This will provide also the required head for energy production. Open canal or buried pipe can be installed as the headrace system.
- **Forebay:** The forebay is a basin located just before the entrance to the penstock. Water is stored and regulated in the basin for the proper and undesired operation of the scheme. The volume of the forebay basin should be enough to cope with water demands created by a sudden increase in loading on the turbine. If the headrace system is designed as pressurized pipe the forebay is replaced by surge chamber.
- **Penstock:** The penstock is a pipe that conveys water under pressure from the forebay to the power house. This is an essential part of hydropower scheme but its installation and equipments are rather expensive and could account for almost half of total civil works cost in high-head SHP.

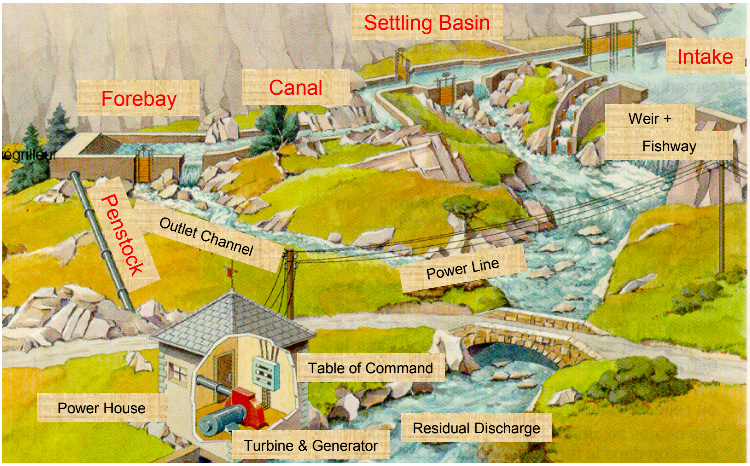


Figure 1.4: Illustration of the principal components of a high-head SHP [32]

The cost of all civil works in a SHP represents a major proportion of the total project cost, often about 50 to 60%.

1.8 Turbines for SHP

Four main kinds of small turbines exist (Figures 1.5, 1.6):

- **Pelton turbines:** with a runner composed of bucket, they fit especially to high-head applications (from 60 m to more than 1000 m).
- **Axial turbines:** with a runner composed of blades, they gather Kaplan, bulbs and propeller ones, and are suited for low head between 2 and 40 m.
- **Francis turbines:** with fixed runner blades and adjustable guide vanes, they are suited for heads from 25 to 350 m [4].

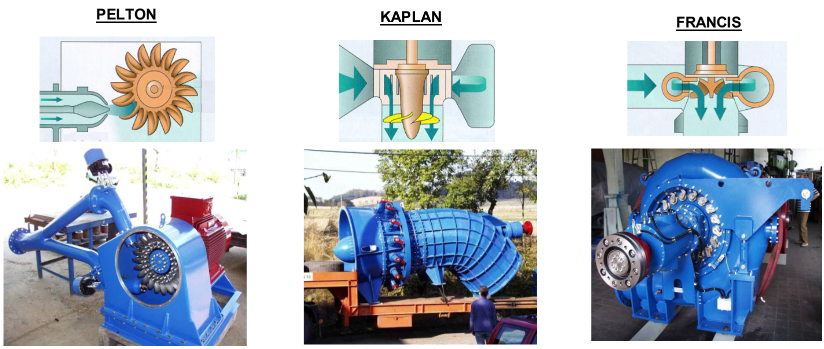


Figure 1.5: View of a Pelton, Kaplan and Francis turbine [28]

- **Cross-Flow turbines:** This impulse turbine, also known as Banki-Michell, is used for a wide range of heads overlapping those of Kaplan, Francis and Pelton. It can operate with heads between 5 and 200 m [4]. Water enters the turbine, directed by one or more guide vanes located upstream of the runner and crosses it two times before leaving the turbine. This simple design makes it cheap and easy to repair in case of runner brakes due to important mechanical stresses. A valuable feature of the Cross-Flow turbine is its relatively flat efficiency curve. This means that at reduced flow, efficiency is still quite high, a consideration that may be more important than a higher optimum-point efficiency of other turbines. Due to low price and good control these turbines are, however, very successful in the area of small hydro-electric power plants.

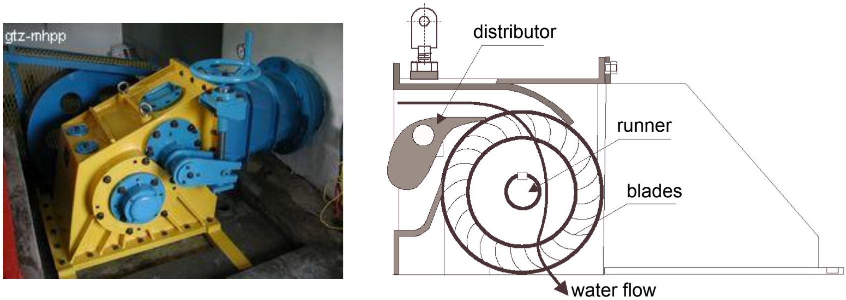


Figure 1.6: View of a Cross-Flow turbine [4]

2 Basic definitions

2.1 Head

Head is the vertical distance that water falls and is a function of the characteristics of the canal or pipe through which it flows.

When determining head, both gross head and net head are considered:

- Gross head (H_{Tot}) is equivalent to the difference between the upstream water level at intake and the downstream water level at tailrace canal after the power house.
- Net head (H_{Net}) equals gross head minus losses due to friction of waterway system.

2.2 Hydraulic power

The hydraulic power or capacity is a function of the design discharge and the available gross head at an assumed global efficiency which includes head loss of different equipments, as shown in the following equation:

$$P = \rho_w \cdot g \cdot Q \cdot H_{Net} \cdot \eta \cdot 10^{-3} \qquad (2.1)$$

P: real power (kW) Q: discharge (m^3/s) g: gravity acceleration, 9.81 (m/s^2)

ρ_w: water density, 1000 (kg/m^3) η: global efficiency (%) H_{Net}: net head (m)

The global efficiency is obtained as:

$$\eta = \eta_{turbine} \cdot \eta_{generator} \cdot \eta_{el.system} \qquad (2.2)$$

$\eta_{turbine}$: turbine efficiency (≈ 0.85) $\eta_{generator}$: generator efficiency (>0.9)

$\eta_{el.\ system}$: efficiency of the rest of electrical system that is transformer but not distribution (>0.9)

2.3 Flow duration curve (FDC)

To select the most appropriate hydraulic equipment, estimate its potential and calculate the annual energy output, a flow duration curve is most useful. A program of stream gauging at particular site over a period of years will provide a table of stream discharges, which have to be classified into a useable form.

One way of organizing discharge data is by plotting a flow duration curve (FDC) that shows the proportion of time during which the discharge equals or exceeds certain values for a particular point on a river. It can be obtained from the hydrograph by organizing data by magnitude instead of chronologically. If the individual daily flows for one year are organized in categories, then a graph like Figure 2.1 will be obtained.

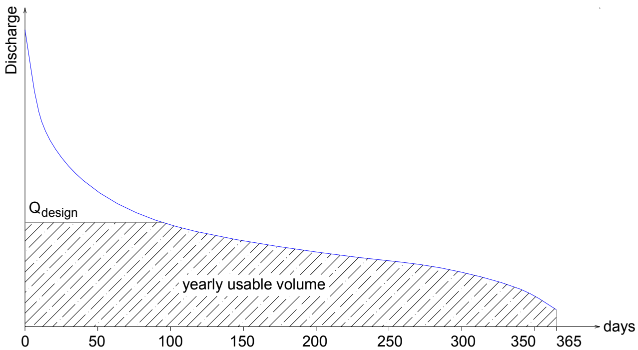


Figure 2.1: Typical flow duration curve

According to the design discharge of SHP, the yearly volume of turbined water will be obtained as shown in Figure 2.1 (sub surface of the flow duration graph represent the volume).

2.4 Energy production

The energy production of the scheme is proportional to the product of the flow and head. The generated electric energy can be estimated by the following equation:

$$E = \rho_w \cdot g \cdot Vol \cdot H \cdot \eta / (3600 * 10^3) \tag{2.3}$$

E: annual energy production (kWh/yr) Vol: yearly mean volume of turbined water (m^3/yr)

2.5 Residual discharge

Small hydro power plants ensure a minimum flow downstream of intake called residual discharge or reserved flow. It is meant to:

- assure survival of fishes and related aquatic lives
- feed the underground water
- favor the species diversity
- valorize landscape
- work alluvial spaces
- assure the cleanliness of water streams

2.6 Turbine minimum discharge

When the river dries up, the flow should not fall below some predetermined amount as the minimum technical flow for turbine operation. This limit depends on the turbine types and specifications and varies between 10 to 30 % of the turbine design discharge [15] (Table 2.1).

Table 2.1: Characteristics of different types of turbines (efficiency and minimum discharge)

Turbine	Type	Maximum efficiency	Minimum turbined discharge	Net head until:
Pelton	action	84 to 90%	**10% of Q_{design}**	Axis of jet
Francis	reaction	84 to 90%	**30% of Q_{design}**	D/S water level
Kaplan	reaction	84 to 90%	**20% of Q_{design}**	D/S water level

In some cases and according to the river characteristics, this limitation of turbine minimum discharge can be reduced.

3 Objective of the present study

3.1 Introduction

For development of small hydropower plants, the lack of simple and generally applicable procedures and methods of realization is observed. The interest of small hydropower resources is increasing but few published guides exist for the optimum design of such projects.

This study aims at providing a general guidance with regard to the economical design and the practical realization of the most important components of such schemes. The main purpose is to standardize the design of the essential components of high-head small hydropower plants for the civil engineering works. It points out those solutions and methods that are thought to be more economical and easily applicable under the given circumstances. The present standardization efforts also include the addition of worked-out examples like drawings and specifications. These factors should represent a significant first step in fulfilling the objective of the project. This kind of efforts at standardization can become a guideline for the designers to better realize and implement the project.

3.2 Main objectives

The main objectives of the present study can be summarized as follows:

- Standardization of the main structures of a small hydropower plant as a function of the design parameters, like discharge and head, with focus on high-head schemes.
- Implementation of the standardized structures in a general applicable optimization tool for the layout of the hydropower plants.

3.3 Phase of the work

For the standardization of small hydropower plants different phases have been considered:

3.3.1 Design criteria for civil works

Hydraulic and basic structural design of the main component of SHP:

- Intake (Bottom intake)
- Settling or desilting basin (sand trap)
- Headrace canal or pipe (open air and buried)
- Forebay or surge tank
- Penstock with anchor blocks

3.3.2 Standardization of structures

Standardization of the principal structures as a function of the design parameters:

- Standardized geometries and constructions details
- Parametric design charts with concrete volume, formwork, excavation and reinforcement
- Construction cost functions (based on design parameters and unit prices)

3.3.3 Implementation of optimization tool

- Review and generalization of existing optimization tool “POPEHYE”
- Implementation of standardized structures and cost functions in optimization strategies
- Derivation of general rules for optimization strategies

3.3.4 Environmental impacts

- Different environmental impacts due to energy production

According to the previous steps, the flow chart of study procedure has been shown in Figure 3.1. Table 3.1 shows the detail of standardization procedure for the main components of SHP.

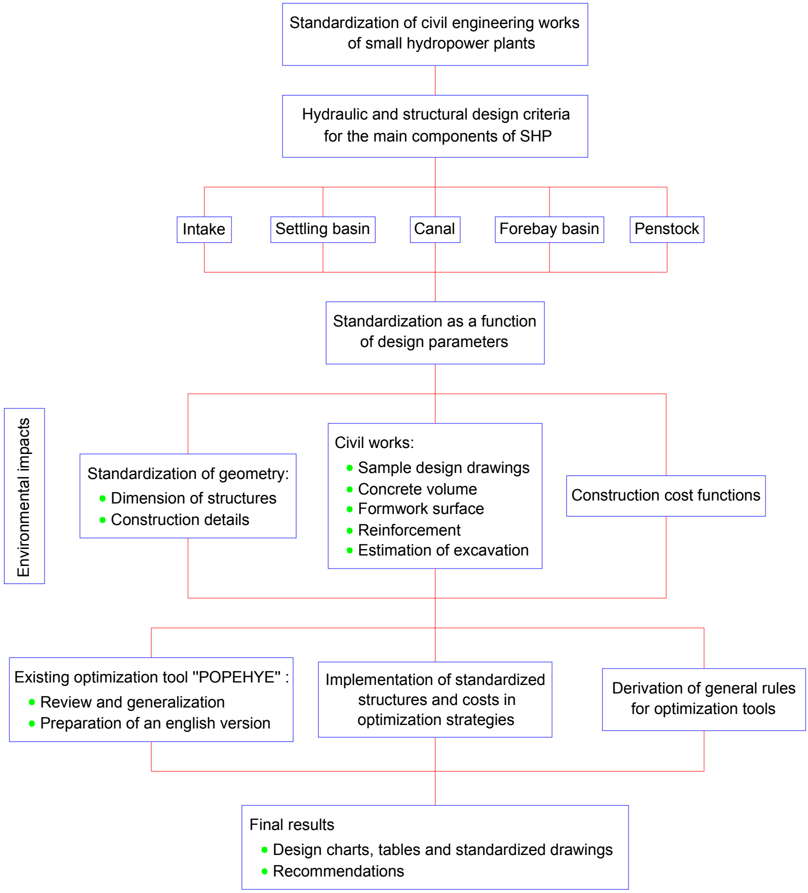


Figure 3.1: Flow chart for standardization and optimization of small hydropower plants

Table 3.1: Summary of standardization and optimization process for different components of a SHP

Hydraulic structures	**Type**	**Section**	**Structure material**	**Standardization***	**Optimization****	**Related structures**
1. Intake	Tyrolean	Trapezoidal	Concrete	+	+	Side walls Access road
2. Settling basin	Bieri flushing system Büchi flushing system	Rectangular + trapezoidal	Concrete	+	+	Access road
3. Headrace system	Open canal Buried pipe Buried canal	Rectangular Trapezoidal Circular	Concrete PVC Rock	+	+	Access road
4. Forebay	Forebay Surge tank	Rectangular Circular	Concrete	+	+	Side spillway Access road
5. Penstock	Open pipe Buried pipe	Circular	Steel PVC	+	+	Anchor block Support Expansion joint

* Concrete volume, reinforcement, formwork, excavation, backfilling ** Cost functions, software POPEHYE

4 Standardization of civil structures

Standardization of civil works of high-head small hydropower plant covers the typical design charts of the various components which contribute to a fast evaluation and comparison of different alternatives of small hydro in the pre-feasibility stage of a project.

This section briefly describes the function of each component and some typical design criteria. Then the main results of geometrical and volumetric design charts will be presented. After estimation of the volume of all civil works for different structures (concrete volume, reinforcement, formwork, excavation, backfilling), the designer can obtain the cost functions according to the unit prices, which will be used later on in the optimization process.

If the list of civil works unit prices are considered as given at Table 4.1, the corresponding cost function based on design discharges can be obtained. It is clear that these unit cost values are variable and that the designer should estimate the total cost according to its own unit values with respect to volumetric design charts of civil works.

Table 4.1: a) Reference unit costs of civil works; b) Additional percentage (study and site installation)

concrete volume [m^3]	reinforcement weight [kg]	formwork surface [m^2]	excavation rock [m^3]	excavation alluvium [m^3]	backfilling [m^3]

a)

project study [%]	site installation [%]

b)

4.1 Water intake

Intakes are defined as structures to divert water into a waterway, such as a headrace canal or a pressure conduit leading to the power plant. A water intake must be able to divert the required amount of water into the conveyance system without producing a negative impact on the local environment and with the minimum possible head loss. Its design and location are based on geological, hydraulic and sediment flushing and also on structural and economical considerations. The water intake should be equipped with a trashrack to minimize the amount of debris and sediment carried by the incoming water.

The Tyrolean or drop intakes are commonly used for small and steeply sloped mountain rivers with reliable rock foundation. This type of intake, used in the absence of a reservoir, consists essentially of a channel built in the river bed, stretching across it and protected by a trashrack with a sloping face oriented in the river flow direction (Figure 4.1).

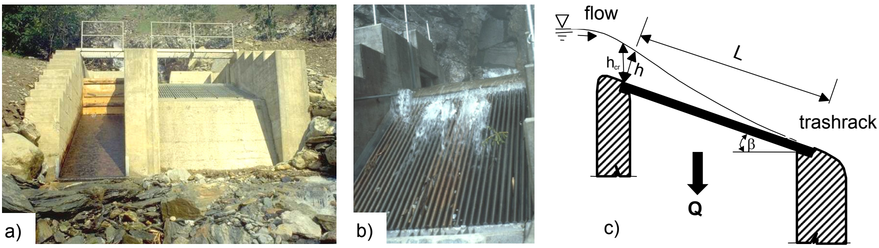


Figure 4.1: a) Tyrolean intake; b) Trashrack; c) Typical profile

In France EDF has improved this type of intake, placing the bars as cantilevers to avoid the accumulation of small stones commonly entrained by the water [8].

The advanced concept of the drop intake is the Coanda-effect screen that offers potential for economically screening fine materials with a minimum of clogging and cleaning maintenance. This self-cleaning screen with no moving part has been successfully used for debris and fish exclusion (Figure 4.2). The screen is typically installed in the downstream face of an overflow weir. Screening capacities of 0.09 to 0.14 m^3/s per meter of weir length have been reported [13]. These screens utilize standard wedge-wire screen panels in which the top surface of each wire is parallel to the plane of the complete screen. Flow passing through the screen is collected in a conveyance channel below the screen, while overflow, debris, and the fish pass off the downstream end of the screen. The screen is capable of removing 90% of the solids as small as 0.5mm, so a silt basin and sediment ejection system can be omitted.

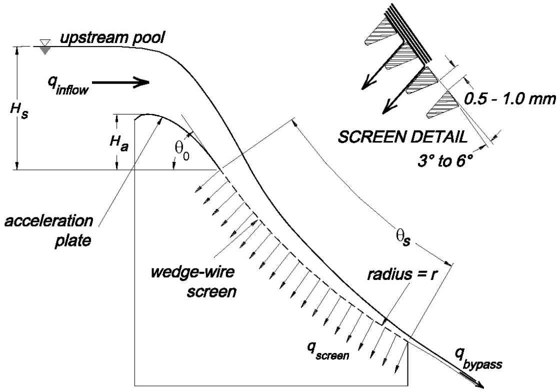


Figure 4.2: Typical arrangement and design parameters for Coanda-effect screens [13]

4.1.1 Location

The location of the intake depends on a number of factors, such as submergence, geotechnical conditions, environmental considerations, sediment exclusion and ice formation where necessary. Drop intakes do not seem to require a bend of the river and a lateral entrance with vertical side track. Suitable sites as for instance narrow gorges with sound rock or reliable foundation materials on which to place the intake structures offer convenient and economical solutions. It is evident that drop intakes are particularly well suited for locations in steep terrains.

A small basin is normally excavated in the upstream river bed of bottom intake to produce a more regular and turbulence free subcritical flow approach, upstream of the rack (Figure 4.3a). This little basin could also be excavated in downstream part for fishes. However, the location of the intake can also be selected in a way that a natural small pool is provided in upstream part. This is possible when the intake is selected according to the nick points in the river bed slope (Figure 4.3b). In the absence of a rudimentary stilling basin, the approach velocity, particularly during flood will be considerable and the flow pattern will probably be quite erratic. In this case some of the flow may then completely bypass the intake and significant loss of water will occur during flow passing over the trashrack.

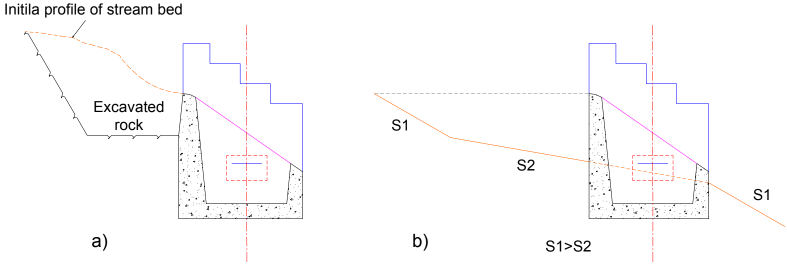


Figure 4.3: a) Stilling system above the intake; b) Proper location of intake due to river slopes

In addition, ready access to the intake for maintenance and repairs, to bring in concrete, replacement bars and other items is essential and should be considered at the same time as the other design factors. Whenever possible, the design should include convenient horizontal platforms, arranged as a stairway for necessary access.

In many countries residual flow is regulated by a national law that is based on hydrological studies and flow duration curve. This is a discharge that has to be released downstream of the intake structure in a river for environmental reasons. Once the residual discharge is defined, the hydraulic device (an orifice) ensuring the achievement of this target must be implemented in the intake design.

4.1.2 Trashrack

Trashrack function essentially as filters to remove material and stones, floating on or just below the water surface from entering to the intake. In design of trashrack type and dimensions, precautions should be taken to prevent clogging of the racks. The basic design variables of trashrack are the opening between adjacent bars "a" and the center spacing "b". These values for SHP depend on the size of materials allowed to pass through the intake. Figure 4.4 shows some types of rack bars with different profiles. The rectangular racks are not recommended to be used for intake as they are easily and rapidly clogged by stones (Figure 4.4a). The bulb-ended bars have better performance and are more rigid if required (Figure 4.4b). Finally the best shape is round-head bars that prevent sediments from jamming and have better resistance against impact of stones because of higher moment of inertia (Figure 4.4c). So, this last type of bars should be used systematically for Tyrolean intake. The recommended opening rack bars for this type is 20 to 40 mm.

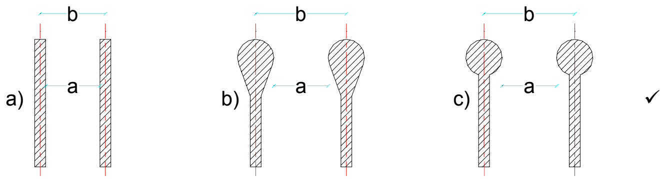


Figure 4.4: Some typical profile of racks for Tyrolean intake: a) to be avoided; b) better; c) the best

The racks have to be inspected and cleaned at regular time intervals to prevent potential obstruction by debris.

4.1.3 Design criteria

The dimensions of the Tyrolean intake must be sufficient to capture all the water for the design discharge. According to shape of trashrack, head losses, approach flow regime and design discharge, the dimensions are based on the following formula [1]:

$$Q = \frac{2}{3} \cdot c \cdot \mu \cdot B \cdot L \cdot \sqrt{2 \cdot g \cdot h} \tag{4.1}$$

B: intake width (m), $B \cong 0.6 \cdot L$

h: water depth at upstream end of trashrack (m), $h = k_c \cdot h_{cr}$

Q: design discharge (m^3/s)

c: trashrack coefficient, $c = 0.6 \cdot (a/b) \cdot (\cos\beta)^{3/2}$

h_{cr}: critical water depth (m)

k_c: Correction factor, $k_c = 0.88 \cdot (\cos\beta)$ for $\beta > 30^\circ$

μ: discharge coefficient, 0.8

β: slope angle of trashrack (°), $30^\circ < \beta < 45^\circ$

L: intake length over the trashrack (m)

In this study, the dimension ratio of rack bars "a/b" are 1/3, 1/2, 2/3 and the trashrack angle "β" is between 30° to 45°, according to most common design practice and experiences. The intake length is normally extended by 20% because of potential obstruction.

4.1.4 Standardization charts for drop intake

The standardization of water intakes considers different combinations of geometry and river bed type. The geometry design charts are presented according to "a/b", "β" and discharges. Figure 4.5 shows the geometrical functions of a Tyrolean intake including width and length for β=35° and a/b=1/2.

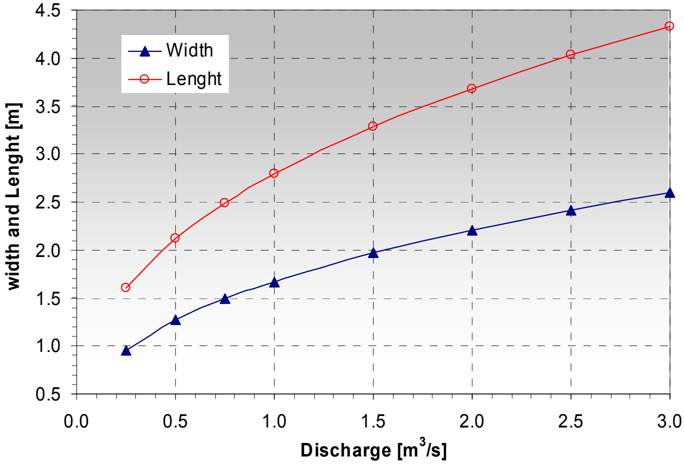


Figure 4.5: Design charts for Tyrolean intake (β=35°, a/b=1/2): required width and length of trashrack as a function of discharge (m^3/s)

The volumetric design charts are presented according to the river width and a series of discharges. As an example, the concrete volume, reinforcement, formwork and excavation of the intake and other related structures (drop intake and side walls) for β=35°, a/b=1/2 and a rocky river bed are shown graphically in Figures 4.6 to 4.9.

<u>All of the standardization results concerning Tyrolean intake are presented in Appendix A.</u>

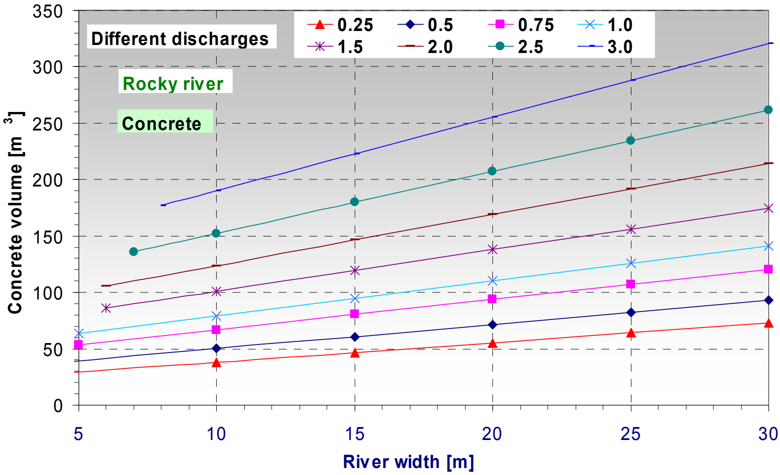


Figure 4.6: Design charts for Tyrolean intake (β=35°, a/b=1/2): required concrete volume as a function of river width and discharge (m^3/s) for rocky bed

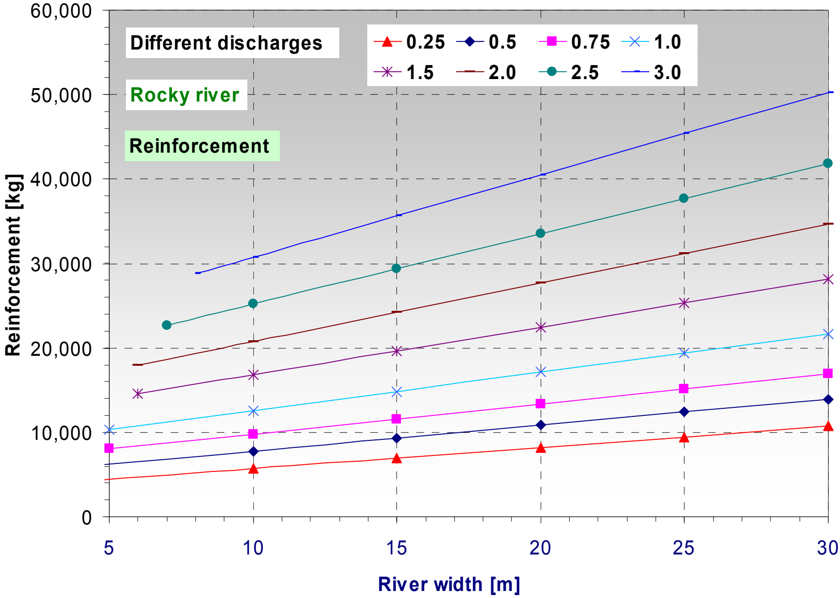


Figure 4.7: Design charts for Tyrolean intake (β=35°, a/b=1/2): required reinforcement as a function of river width and discharge (m^3/s) for rocky bed

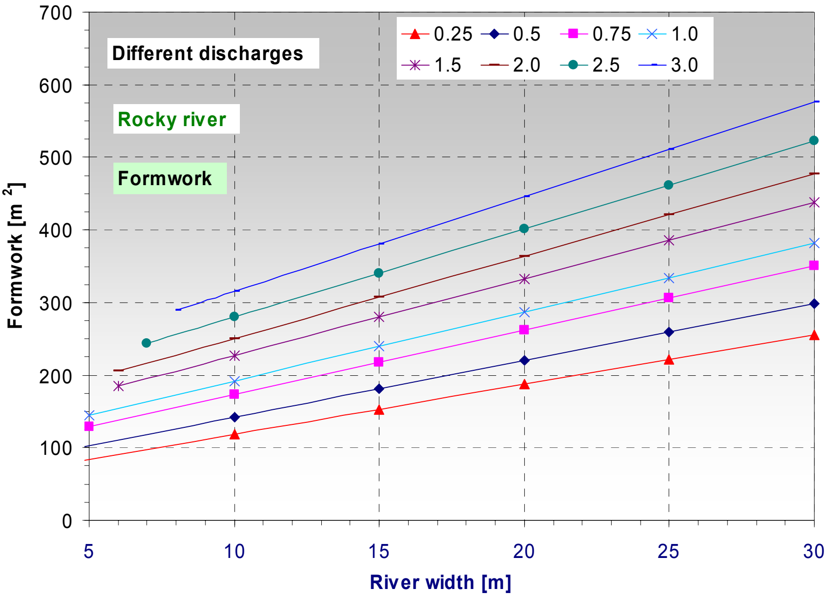


Figure 4.8: Design charts for Tyrolean intake (β=35°, a/b=1/2): required formwork as a function of river width and discharge (m^3/s) for rocky bed

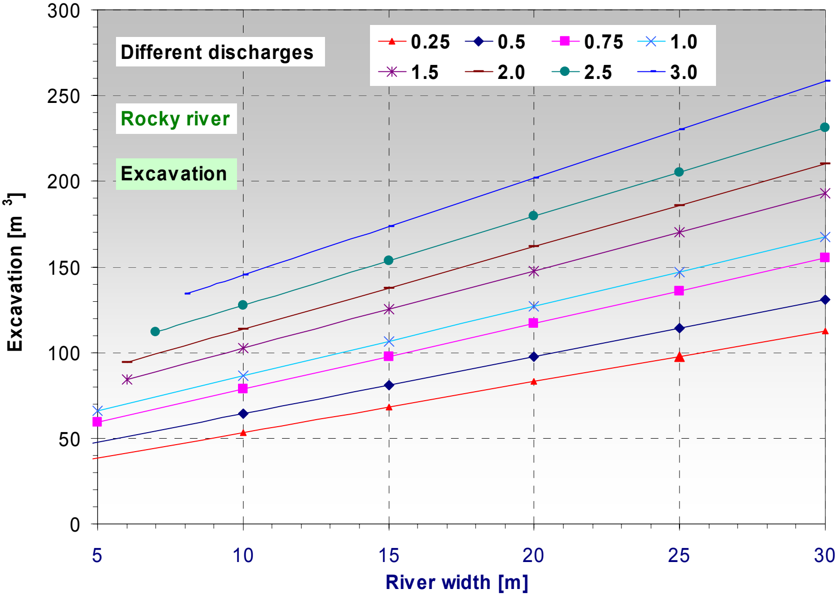


Figure 4.9: Design charts for Tyrolean intake (β=35^o, a/b=1/2): required excavation as a function of river width and discharge (m^3/s) for rocky bed

4.2 Settling basin (sand trap)

The settling basin is located downstream of the intake and permits to separate the sand from the incoming water and to prevent them from settling in the headrace conduit or being carried through the turbine, which causes abrasion and a decrease of the turbine's life and efficiency.

The longitudinal settling basin consists of one or two chambers of sufficient length with sloping bottoms to allow the sediment particles to settle down. Before entering the main basin, water will pass through a transition part with stilling rack system for creating a uniform flow distribution. In the high mountains the desander basins have to be protected in certain cases against avalanches and stonefalls. They can be built into caverns or covered with concrete plates. In lower regions, the desanders do not have to be covered.

For the purpose of this study, two well known types of settling basins, the Bieri and Büchi flushing systems have been selected.

4.2.1 Bieri flushing system

In Bieri type the sediments which settle in the basin are flushed vertically through the opening into the channel and back to the river [1] (Figure 4.10). The flushing water volume is therefore minimized and energy production is ensured even during the flushing procedure. For about 50 years, more than 80 Bieri flushing systems have been in operation around the world [19].

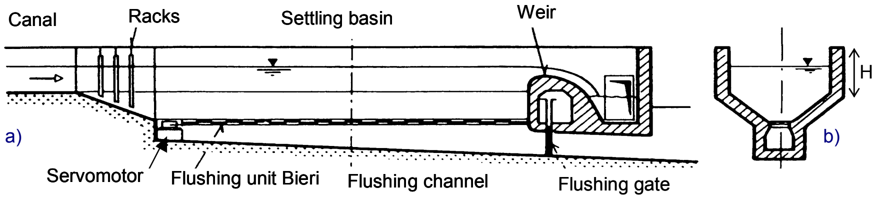


Figure 4.10: Layout of Bieri sediment trap: a) Longitudinal section; b) cross section [1]

4.2.1.1 Basin unit

The water flows through the basin in the longitudinal direction, and the overflow structure at the end of the basin maintains an adequate water level, and regular flow through the basin. The sand flushing unit is installed at the bottom of the V-shaped desander basin (Figure 4.11). An inlet gate is necessary for drainage of the desander for inspections and maintenance work and is fully opened during operation. The sand flushing gate also serves as an emergency closing gate in the event of any of the flushing units becoming blocked in the open position.

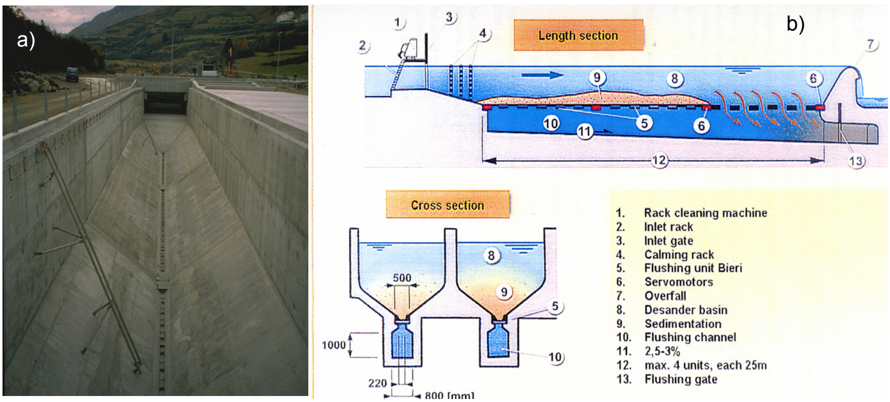
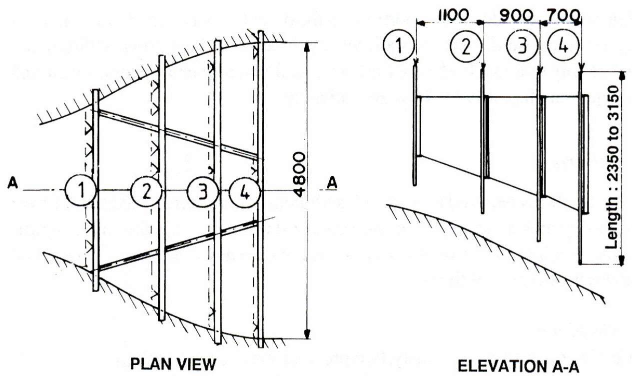


Figure 4.11: a) Longitudinal view of a Bieri system; b) Different components of Bieri flushing system [19]

4.2.1.2 Stilling racks and transition channel

The flow velocity in a channel leading up an intake to a sand trap is fixed by the need to prevent sediment deposition. This flow velocity is typically 1.5 to 2.0 m/s. Ahead of a settling basin, a gradual expansion is generally provided to reduce the velocity to about 0.2 m/s. A stilling rack at this transition zone is provided to damp down any turbulence at the entrance to the settling basin. Figure 4.12 shows the stilling rack system that should be installed ahead of the trap. The design depicted is not arbitrary. The optimal system includes upstream-facing angle bars whose size and spacing diminish in the downstream direction so that the resulting vortices become progressively smaller and thus release quickly their energy. The angle bars should never extend to the apron because it would be virtually impossible to remove the stones that might be retained.



1	9 angle iron bars,	60x60x6,	length 2000		spacing 280
2	17 ,, ,,	50x50x5,	,, 2000	to 2400,	,, 215
3	27 ,, ,,	40x40x4,	,, 2180	2800,	,, 165
4	39 ,, ,,	35x35x4,	,, 2350	3150,	,, 120

Figure 4.12: Schematic of stilling rack system (dimensions in mm) [7]

4.2.1.3 Sand flushing unit

This unit consists of two apertured plates, one on the top of the other, which are horizontally arranged against each other on the bottom of the basin (Figure 4.13a). The lower plate is fixed rigidly to the structure, and the upper plate is moved by hydraulically operated servomotors (Figure 4.13b). When the flushing unit (the upper part) is opened, the flushing operation takes place through an opening of approximately 50 * 200 mm. In the completely opened position, the size of the opening is 190 * 200 mm. However, as mentioned above, for regular flushing, the openings are only approximately 50 – 70 mm and are determined after the installation at the site. The values are adjustable and depend from case to case. The height and width of the flushing channel are recommended as 1.0 and 0.8 m respectively [19].

The units of the sand flushing device are built in standard lengths of 20, 25 and 30 m. As mentioned above, in normal conditions, the sluice gates are only opened partially. Thanks to the sand level measuring sensors in each unit, only those desander units in which sufficient sand is deposited are opened. The newly plastic coated Motec-plates (Figure 4.13c) in the new sluice construction reduce leakage in the system by another 90% [19].

For installation of the unit, a recess inside the basin over the whole length has to be foreseen. The basis structure of the desanding unit is welded onto reinforcement irons of the basin structure and the recess is poured out with concrete (Figure 4.13d). No special recesses are required for the servomotors.

The flushing is initiated automatically with a timer at intervals of 0.5 to 12 hours, or by measurement of the height of the accumulated sand. Scanning can also be done by a “vega lot” system [19]. This means that only a minimum of flushing water is used, and there is almost no interface in the production capacity of the turbines. The automatic control system indicates any interruption on a control panel, and these can be transmitted to the power plant’s control room. Also the desanding control system is fully automatic; it can also be manually activated in cases of emergency, or for special purposes. The automatic flushing process should be done during the night when there are few people in downstream part of the river.

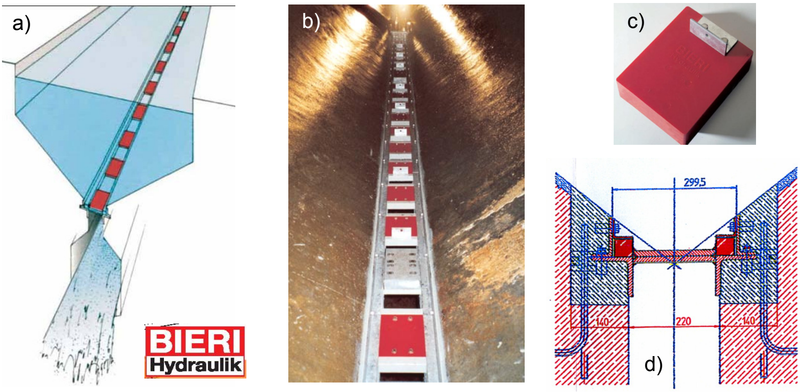


Figure 4.13: a) typical section of Bieri basin and unit; b) sand flushing unit; c) plastic coated Motec-plate d) installation detail of the units (mm) [19]

So the following values can be regulated by the control system of the Bieri desander:

- volume of sand deposit
- number of flushing intervals
- slide opening
- duration of flushing time

4.2.1.4 Advantages of Bieri flushing system

The Bieri desander has the following advantages in comparison to other similar systems:

- Considerable saving of flushing water, i.e. the water is used for power production rather than flush sand.
- Less danger for river in downstream parts
- No continuous flushing water requirement.
- Amortization of the whole installation within a short time.
- Only the units with the pre-determined volume of sand deposit are flushed (the flushing are performed in steps with only one unit being opened at a time).
- Adjustable for each individual power station since sand quality, sediment composition and contents are different on each project.
- No clogging of desanding system. Materials which may pass the intake rack are sheared off between the aperture plates.
- The flushing system is self cleaning.
- Higher overall performance of the turbine system and therefore of the whole power plant.

4.2.1.5 Comparison of Dufour with Bieri flushing system

In Dufour sediment trap, continuous sluicing of the sediment deposits is achieved by means of openings in the chamber bottom through which the sediment together with a certain amount of water enters a channel with a flushing gate at its downstream end (Figure 4.14).

The sediment exclusion system is constructed of wood, consisting of 4m long modules each having two orifices about 10cm height, formed by gaps between succeeding planks of wood, 2m long and inclined slightly to the horizontal. The width of orifices varies from 20cm at the upstream end of the excluder to 10cm at the downstream end. The gutter underneath removes the outflow from the excluder, at a velocity on the order of 2 - 2.5 m/s. The exclusion flow usually amounts to 5 or 10% of the flow entering the trap. It is also controlled by an outlet valve [7].

Thought this type of sand trap operates satisfactorily, a problem arises in that the larger material settles out at the upstream end of the exclusion system whereas the fines appear nearer to the outlet. This means that the scour flow has to move the larger items much further than the fines.

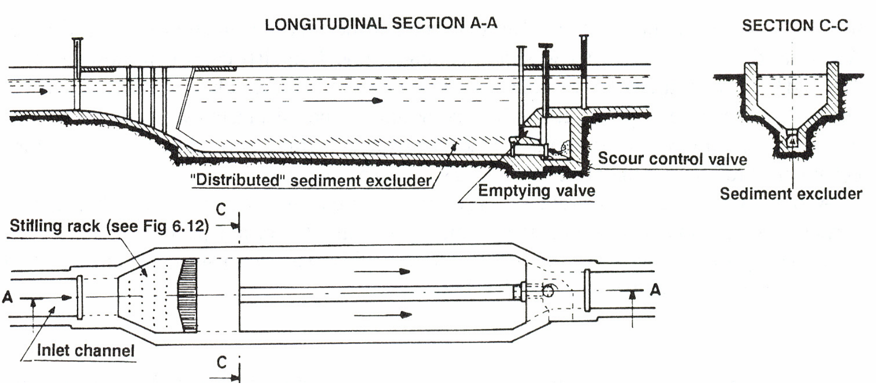


Figure 4.14: Schematic of Dufour sand trap [7]

The cross section of a Bieri sand trap is similar to that of a Dufour type but they operate rather differently. The Dufour scour flow is controlled by a single outlet valve, which usually has to be at least half open. The Dufour scour orifices are quite large (15 * 15 cm) because there is only one every 2m [7]. Their role is to distribute the washout flow over the length of the trap and not to control the amount of this flow. The outlet valves and scour orifices are then not likely to clog.

In a Bieri system, on the other hand, the flow in the scour channel is not controlled by a valve and the scour orifices operate under the full head. The oil jacks provided to adjust the size of the orifices can be made powerful enough to cut through probable clogged debris. Alternatively, the scour orifices can be opened to the maximum size of 19 * 20 cm to flush through the larger items of debris. Bieri trap with flushing system has however two advantages. One is that the washout flow is uniformly distributed, at least along each length of flusher. The other is that the sediment outflow duct is readily accessible for inspection.

According to the above, the Bieri flushing system is chosen for standardization process, because it is considered to offer better performance for sediment control and flushing. Of course civil works for all types of sediment traps are not very different.

4.2.2 Büchi flushing system

For the Büchi type, the separate flushing channel of Bieri system has been removed and the sediment flushing is done by drawdown of the settling chamber periodically (Figure 4.15).

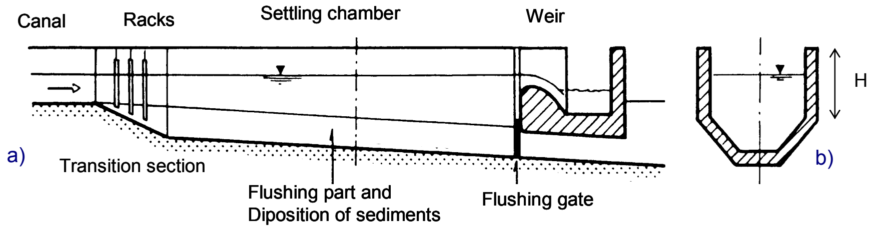


Figure 4.15: Layout of Büchi flushing system: a) longitudinal section; b) cross section [1]

Büchi settling basins operate in quite a different manner from the sand traps discussed above. Sediments therefore start depositing at the inlet. That reduces the flow section, increases the velocity, and reduces the settling efficiency. If the tank was allowed to fill completely, all the incoming sediment would continue on downstream. When the load deposited in the basin becomes critical and unacceptably large sediment loads tend to be entrained downstream, a sluice gate can be opened at the end of the tank to remove the material deposited. The slope of the basin floor and the size of the gate are defined in a way that the scour flow velocity should be considerable. Anyway, the flushing procedure takes a certain time, during which no water can be diverted through the intake.

This conventional sediment trap with horizontal flushing remains full of water during energy production (Figure 4.16a) but has to be drained almost completely whenever it is flushed out (Figure 4.16b). Note also that emptying the basin and washing out the sediment gives rise to a sudden surge of flow below the basin. This is one of the factors that limit the design discharge to 4-5 m^3/s. Several basins can be arranged in parallel to increase the flow handled.

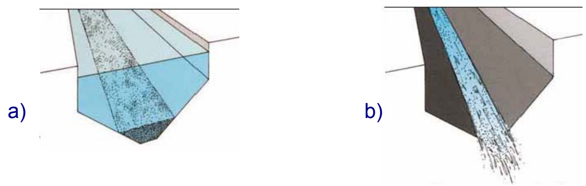


Figure 4.16: a) Normal operation of Büchi flushing system; b) Complete drainage of Büchi basin [19]

4.2.3 Design criteria

The maximum grain size of the sediments to be excluded determines the design feature of the sand trap. As far as small hydropower schemes are concerned, the design grain size of the settling basin is typically 0.2 or 0.3 mm. The length, width and depth of the chamber must fulfill the following conditions [1]:

$$L \geq \frac{Q}{V_D \cdot B} \quad ; \quad V_T < V_{cr} \quad ; \quad B \leq L/8 \quad ; \quad B \leq 2h \text{ (typically 1.5h)} \qquad (4.2)$$

B: basin width (m)

Q: design discharge (m^3/s)

V_D: settling velocity in flowing water (m/s)

V_T: mean flow velocity in basin (m/s)

V_{cr}: critical flow velocity in basin (m/s)

h: water depth in basin (m)

Critical velocity of flow in the basin is normally around 0.2 m/s and found as:

$$V_{cr} = K \cdot R^{1/6} \cdot \sqrt{0.03 \cdot (\frac{\rho_s}{\rho_w} - 1) \cdot d} \quad ; \quad K=60\ m^{1/3}/s,\ \rho_s/\rho = 2.65 \Rightarrow V_{cr} = 13 \cdot R^{1/6} \cdot d^{1/2} \tag{4.3}$$

K: coefficient of Strickler ($m^{1/3}/s$)

R: hydraulic radius (m)

ρ_s: sediment density (kg/m^3)

ρ_w: water density (kg/m^3)

d: design grain size (m)

The empirical formula of Zanke is used to define the settling velocity [1]:

$$V_{D_0} = \frac{100}{9 \cdot d} \cdot (\sqrt{1 + 1.57 \cdot 10^2 \cdot d^3} - 1) \quad ; \quad V_D = V_{D_0} - \alpha \cdot V_T \quad ; \quad \alpha = 0.132 / \sqrt{h} \tag{4.4}$$

V_{D_0}: settling velocity in still water (mm/s)

d: design grain size (mm)

water temperature: 20°C

α: reduction factor ($1/m^{1/2}$)

h: water depth (m)

$\rho_s/\rho = 2.65$

It is recommended also to extend the computed basin length by 10 to 20 % to compensate the excessive turbulence in the approach flow. For high discharges it is also more efficient to have two parallel basins for flexibility against flushing procedure without taking the power plant out of service, especially for Büchi type sand trap.

4.2.4 Standardization charts for sand trap

Design of a settling basin for a series of discharges and two different sediment grain sizes has been accomplished. The standard design charts include the geometry of the basin, such as length and also volumetric functions of the civil works. These results for the Bieri settling basin and a rocky bed are presented in Figures 4.17 to 4.21. After obtaining all of these functions, the total cost can be estimated according to unit prices. It is clear that for removing the sediment sizes greater than 0.2 mm, the length of the basin is higher than for 0.3 mm (Figure 4.18).

<u>All of the standardization results concerning Bieri and Büchi settling basins are presented in Appendix B.</u>

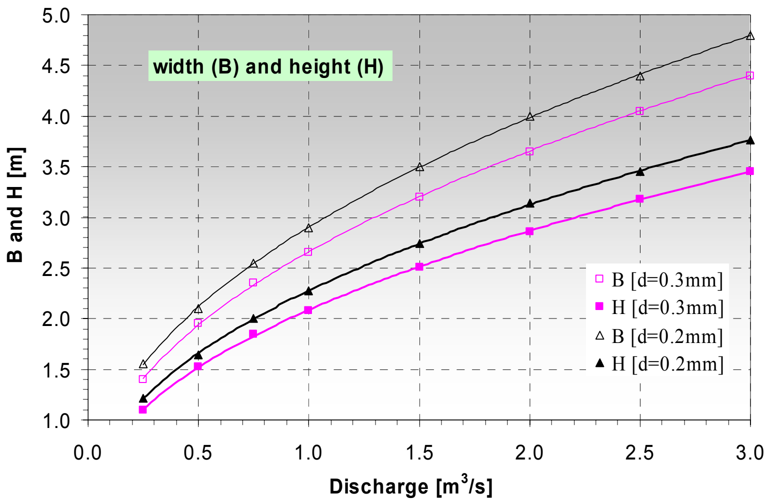


Figure 4.17: Design charts for settling basin: required width and height as a function of discharge [d=0.2 & 0.3mm]

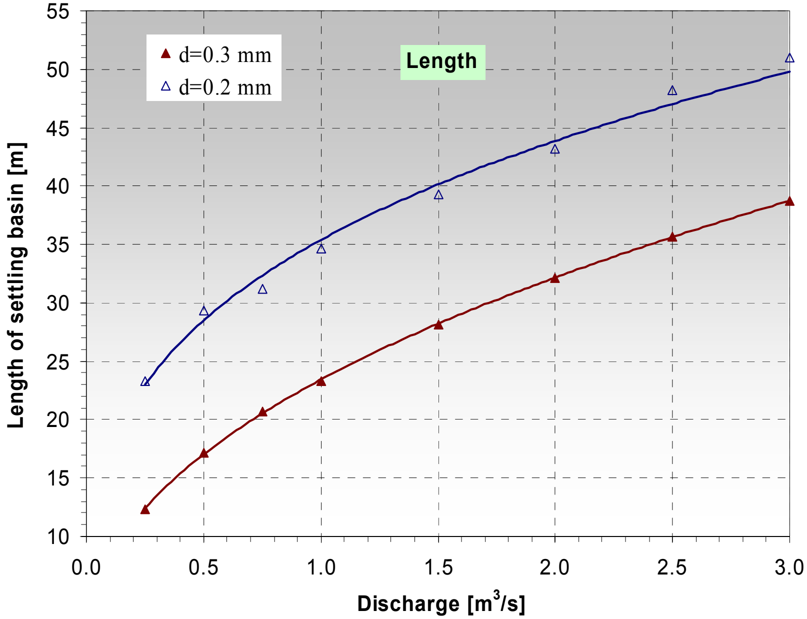


Figure 4.18: Design charts for settling basin: required length as a function of discharge [d=0.2 & 0.3mm]

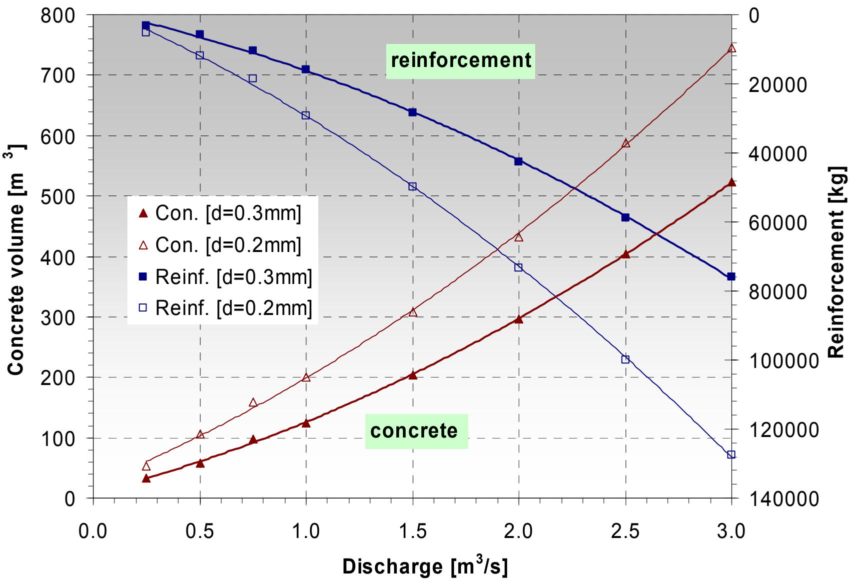


Figure 4.19: Design charts for Bieri flushing system: required concrete volume and reinforcement as a function of discharge [d=0.2 & 0.3mm]

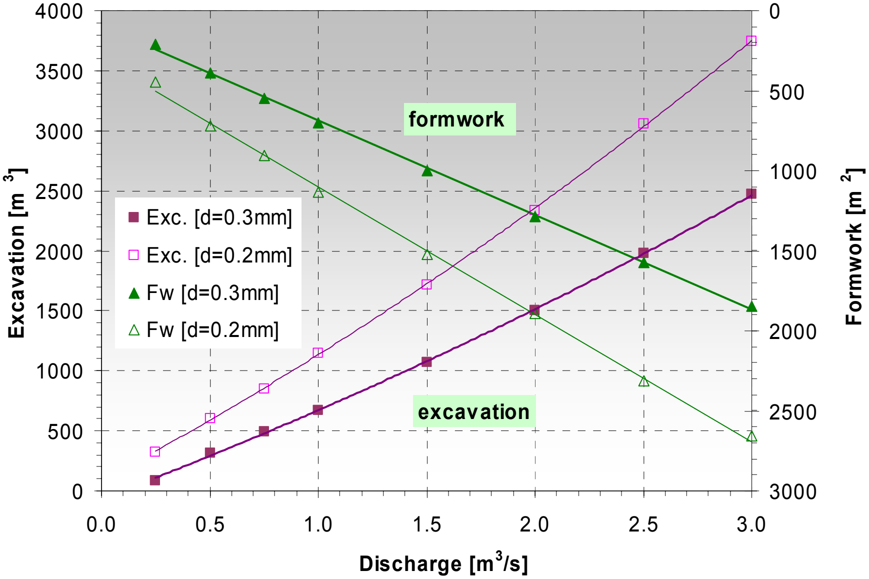


Figure 4.20: Design charts for Bieri flushing system: required excavation and formwork as a function of discharge [d=0.2 & 0.3mm]

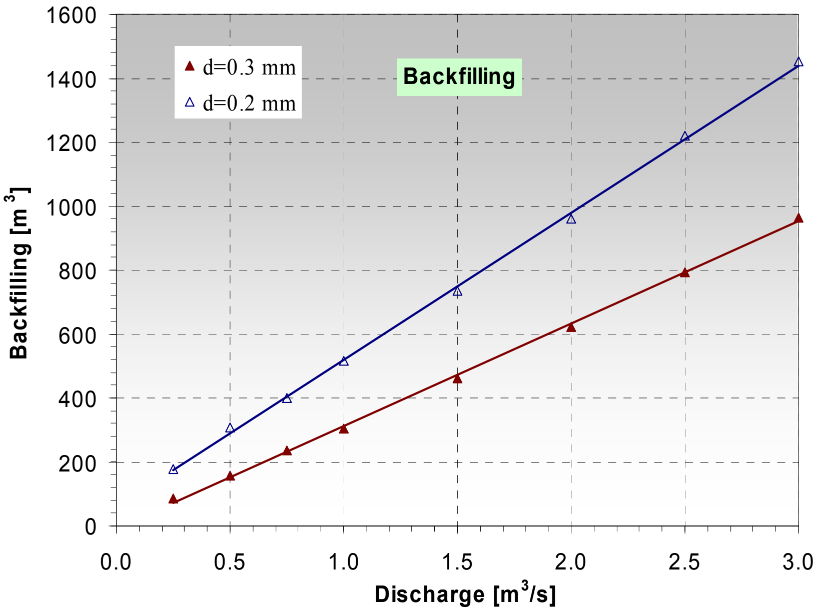


Figure 4.21: Design charts for Bieri flushing system: required backfilling as a function of discharge [d=0.2 & 0.3mm]

4.3 Headrace canal or pipe

The headrace component of a small hydropower plant conveys water from the settling basin to the beginning of the penstock situated in the forebay. The system is most frequently excavated in soil or rock and is usually lined or constructed with concrete and possible backfill or access road (Figure 4.22). The headrace systems should perform their functions efficiently with minimum maintenance, ease of operation and minimum head losses. Canals can be free surface or buried, but pipes are commonly placed underground. Tunnels are also used where it is more economical to convey water through a rock outcropping or hill. There exist different alternatives for headrace conduits like:

- Open canal (lined or unlined)
- Buried canal (lined)
- Buried pipe with free or pressurized flow (concrete or PVC)

Figure 4.22: Examples of open headrace canals

4.3.1 Design criteria

The canal has to be adapted to the natural configuration of the slopes and the topographic conditions. For the range of water discharges studied, canal slopes between 0.1 and 0.4 % have been considered. The higher slope limits should be used for rather long canals to guarantee the flushing process of the whole length. The flow regime must remain subcritical and uniform throughout the length of the canal with free flow. There must be adequate freeboard for open air systems for security and emergency operation. The length and location of the canal is determined with the position of the power plant and a convenient head.

A rectangular cross section is often the most appropriate alternative for a tailrace system because easy to built and maintain. The optimum hydraulic section for a rectangular canal is obtained when the width is about two times the water depth.

Figure 4.23 summarizes the necessary procedure to determine the cross sectional dimensions of a canal and its slope. Each circular number in this Figure refers to the correspondingly numbered paragraph below which explains how to derive the desired quantity following that circle:

1) The cross sectional area of the water (A) in the canal is determined from the basic relation:

$$A = \frac{Q}{V} \tag{4.5}$$

Q: design discharge (m^3/s)   V: design velocity in the canal (m/s)

Normally it is recommended not to exceed a flow velocity of 2.0 to 2.5 m/s in the canal. In addition, a minimum velocity of about 0.6 m/s has to be considered in order to avoid sediment settling in the headrace canal.

2) For the most efficient canal section, the hydraulic radius depends only on the profile selected and the cross sectional area of water it contains. To determine the value of “R” it is necessary to substitute the values of “A” obtained above in the expression in Table 4.2 for the profile selected.

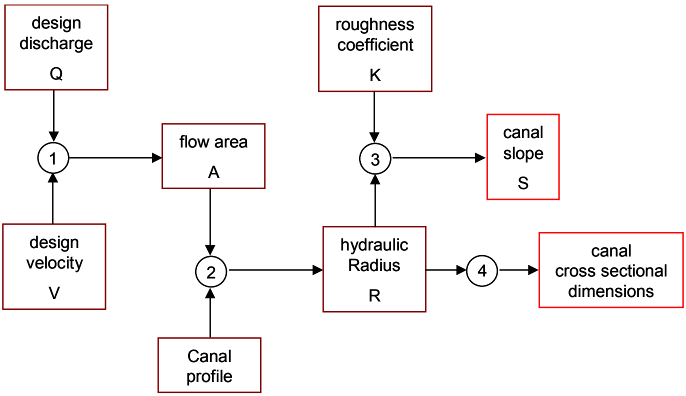
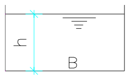
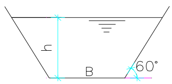


Figure 4.23: Flow diagram of procedure for determining canal dimensions and slope

Table 4.2: Expression of hydraulic radius "R" for the most efficient canal section of common profiles [6]

Profile	Hydraulic radius, R (m)	Cross section
Rectangular	$0.354 \cdot \sqrt{A}$	
Trapezoidal	$0.380 \cdot \sqrt{A}$	

3) Once the hydraulic radius has been determined, the canal slope "S" can be found using Manning-Strickler equation:

$$S = \left(\frac{V}{K \cdot R^{2/3}}\right)^2 \tag{4.6}$$

V: design velocity (m/s)

K: Strickler coefficient ($m^{1/3}/s$)

R: hydraulic radius (m)

S: canal bed slope

Strickler coefficients for lined and unlined open canal are about 65 to 35 respectively.

4) After determining the hydraulic radius, the cross sectional dimensions of the wetted area can be derived using expressions in Table 4.3 for the profile selected. These variables are the water depth "h" and the width of the canal "B".

The procedure described above approaches the sizing of a headrace canal in SHP. Having certain flow requirements at a specific site, the slope is determined, and the canal is laid out according to the topography condition.

Table 4.3: Expression for dimensions of most efficient cross section for common profiles

Profile	Water depth, h (m)	Canal width, B (m)
Rectangular	$2 \cdot R$	$4 \cdot R$
Trapezoidal	$2 \cdot R$	$\frac{A}{h} - \frac{h}{\tan(\theta)}$ *

* θ: angle of the canal banks above horizontal

For very small hydropower plants, for which concrete could be relatively expensive and good rock quality exists in the site, the canal may be excavated directly into the ground and without lining. Care should be taken not to exceed the permissible erosion velocities.

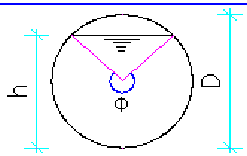
In the high mountains where landsliding and stonefalls could occur during operation of the headrace system, canals have to be protected. They can be built under the ground or just covered with concrete plates. This canal is usually called box culvert.

When the configuration of site and topography condition is not convenient for placing an open canal, the buried pipe could be used as headrace system (Figure 4.24).

Figure 4.24: Typical profiles for buried headrace pipe in construction stage

For the free surface flow regime in the pipe, the hydraulic specifications of the headrace buried system are illustrated in Table 4.4.

Table 4.4: Geometrical properties of circular cross section (free surface flow)

Profile	Flow area, A (m^2)	Wetted perimeter, P (m)	cross section
Circular	$\frac{1}{8}\cdot(\Phi - \sin(\Phi))\cdot D^2$	$\frac{1}{2}\cdot\Phi\cdot D$	

The buried pipe can be manufactures with concrete or PVC which are considered in this study. The canal slope “S” will be found using Manning-Strickler equation as open canal and roughness coefficient of Strickler for PVC pipe is about 85. As design criteria, for free surface flow in pipe, the water depth should not increase more that 85% of diameter.

In some special cases the flow regime in buried pipe could be under pressure. Then, the pipe diameter will be obtained according to design velocity.

4.3.2 Standardization charts for open headrace canal

Canal structures have been standardized for concrete dimensions, formwork, reinforcement and excavation and are appropriately sized to provide for hydraulic, structural and stability design considerations. Between all the components of small hydro, headrace conduits have the highest number of alternatives (Table 4.5).

Table 4.5: Different possible alternatives for headrace systems

Profile	Construction material	Cross section	Flow regime
Open canal	concrete	rectangular	free surface*
	rock	trapezoidal	
Buried canal	concrete	rectangular	
Buried pipe	concrete	circular	
	PVC	circular	
Buried pipe	concrete	circular	under pressure
	PVC	circular	

* Canal bed slope: 0.1, 0.2 and 0.3%

As an example, design charts are presented for an open concrete canal with a bed slope of 0.1 % (canal with short length) and a rectangular section for a series of possible discharges (Figures 4.25 to 4.28).

<u>All of the standardization results concerning different alternatives of headrace system are presented in Appendix C.</u>

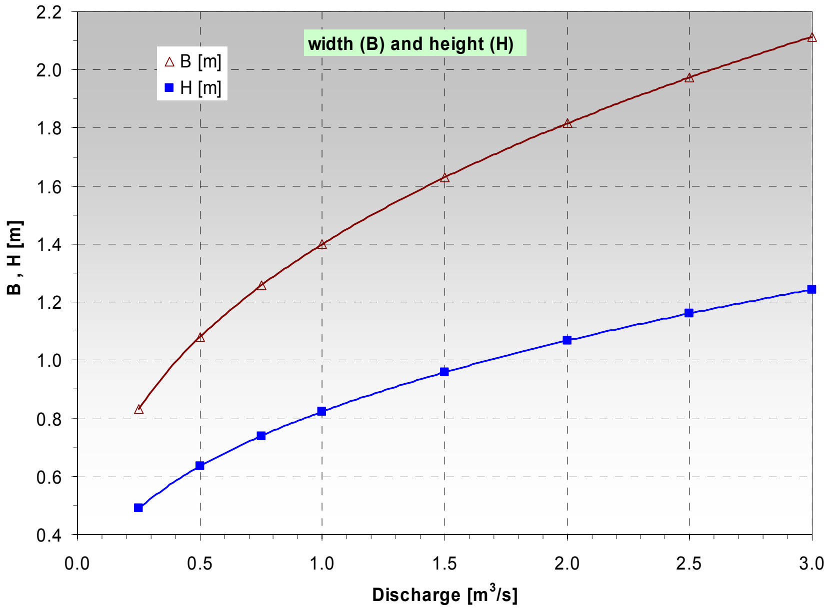


Figure 4.25: Design charts for an open headrace canal (S=0.1%): required width and height as a function of discharge

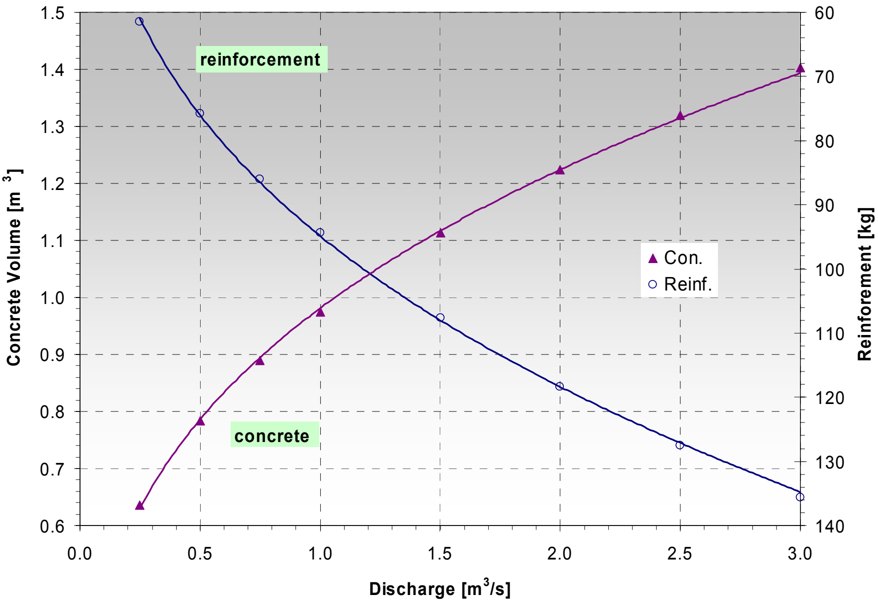


Figure 4.26: Design charts for an open headrace canal (S=0.1%): required concrete volume and reinforcement as a function of discharge <u>for unit length of canal</u>

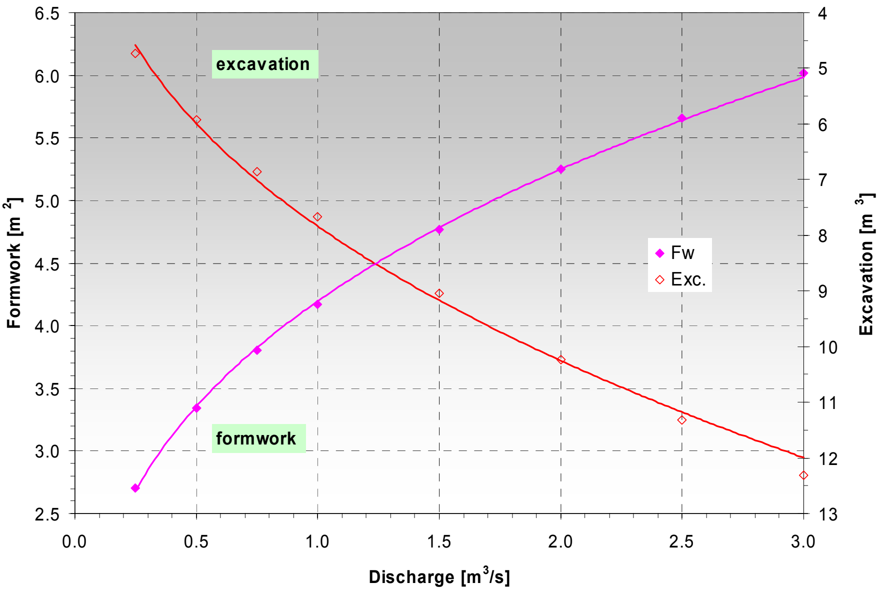


Figure 4.27: Design charts for an open headrace canal (S=0.1%): required formwork and excavation as a function of discharge <u>for unit length of canal</u>

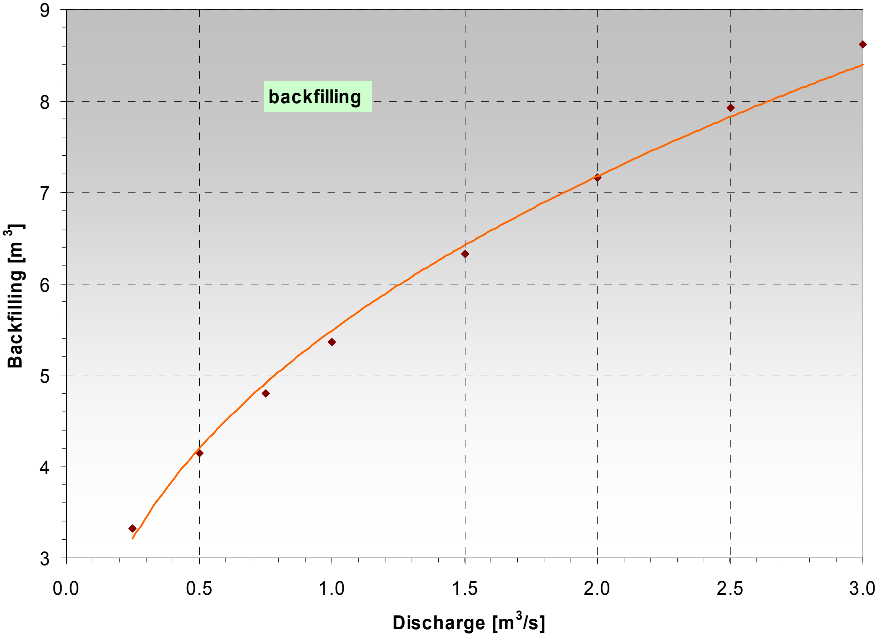


Figure 4.28: Design charts for an open headrace canal (S=0.1%): required backfilling as a function of discharge <u>for unit length of canal</u>

4.4 Forebay

The construction of a forebay downstream of the headrace canal and the entrance of the penstock guarantees not only that no air enters the system, but also allows maintaining a relatively constant head of water into the penstock during transient conditions.

The forebay provides water storage because a significant volume of stored water would be required to supply a turbine for several hours each day. This structure is usually located above the power house and at the top of a steep drop. Therefore, it must be carefully designed and constructed.

However, a forebay generally provides enough storage to cope with water demands created by a sudden increase in loading on the turbine that is a transient condition.

The design of a forebay consists of a transition zone between the end of the canal and a simple basin (Figure 4.29). Its size may vary depending on the required volume storage. It is evident that a forebay should have a minimum cost but its size should be large enough to fulfill its purpose.

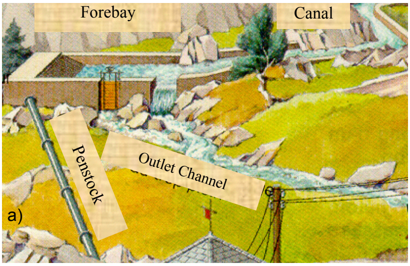


Figure 4.29: a) Schema of a forebay for small hydro; b) A forebay in construction stage

Several important components are usually included in the forebay. These include a spillway, a scouring gate, and a gate to the penstock, a trashrack, and possibly a vent (Figure 4.30).

4.4.1 Spillway

On occasion, the flow entering the forebay may exceed the flow leaving via the penstock. This can occur when the valve to the turbine has been closed or when excess flows enter the canal from the stream or runoff uphill of the canal during heavy rains. For this reason, it is advisable to include a spillway in one wall of the forebay. The water passing over the spillway must be diverted properly to prevent erosion that might undermine the forebay, penstock, or power house.

4.4.2 Trashrack

A trashrack is often included at the inlet to the penstock to prevent possible floating debris from entering penstock and turbine. Skimmers are also used occasionally. If excess water is sufficient, the spillway can be located to remove most of the floating debris automatically.

4.4.3 Flushing gate

A gate or valve should be incorporated to drain the forebay such that any possible sediment which has entered and settled can be removed easily. Draining is also required when the forebay is being repaired. The flow through the drain can be led away in the same canal that removes the overflow from the spillway at the forebay.

4.4.4 Penstock gate or valve

When the penstock must be emptied for repairs, a valve might be incorporated at the beginning of the penstock. However, because such a valve would be the same size as the penstock, it can be costly. In addition, it would only be used infrequently. A less costly approach is to ensure no water enters the forebay, by closing a gate either at the intake to the scheme or just before the forebay (if a spillway is located upstream of that gate).

4.4.5 Air vent

An air vent is often used as a safety precaution against collapse of the penstock pipe. The design would be to incorporate this pipe at the downstream of the penstock valve (Figure 4.30).

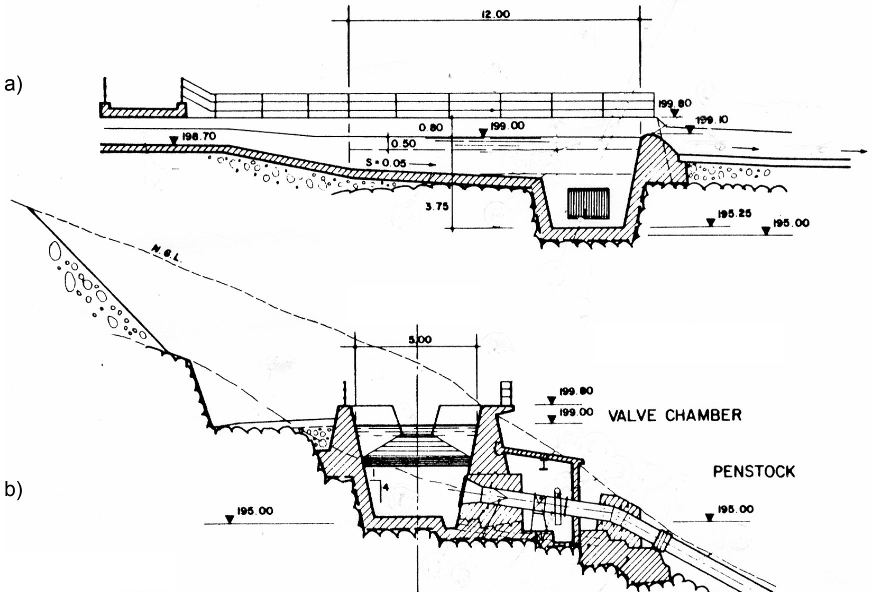


Figure 4.30: An example of forebay design [5]: a) Profile; b) section

4.4.6 Surge tank

A surge tank instead of a forebay is more likely required for high-head SHP when the power conduit is under pressure. The dimensions are obtained based on the assessment of water level fluctuation due to sudden closure or opening of the turbine and also influenced by the head losses and length of the headrace canal or pipe. They are usually open at the top and connected to the penstock pipe at the bottom.

4.4.7 Forebay design criteria

The forebay should provide enough storage of water for starting and regulation of the turbine. For small plants, the required surface area of the forebay can be estimated as a function of design discharge considered here (Table 4.6).

Table 4.6: Recommended forebay area for SHP [5]

Design discharge [m³/s]	Area of forebay [m²]
0 - 3	60 - 120

The required volume of the forebay (the reserve volume above the minimum operation level considering the submergence depth) will be controlled by the following formula:

$$Volume > 2 \cdot T_c \cdot Q \qquad (4.7)$$

T_c: critical time *(refer to 4.4.8.3)* (s) Q: design turbined discharge (m^3/s)

There are different formulas [1] for required minimum depth at the penstock inlet to avoid vortices and air entrainment (Figure 4.31):

Knauss: $h_t \geq D \cdot (1 + 2.3 \cdot \frac{V}{\sqrt{g \cdot D}})$ *Rohan*: $h_t \geq 1.474 \cdot V^{0.48} \cdot D^{0.76}$

Gordon: $h_t \geq c \cdot V \cdot \sqrt{D}$ (c=0.7245 and 0.5434 for non-symmetric and symmetric approach) (4.8)

D: penstock diameter (m) h_t: submergence depth (m) V: flow velocity in penstock (m/s)

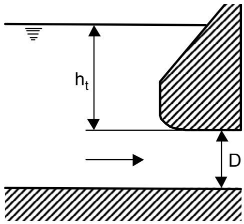


Figure 4.31: Geometric definition of minimum submergence depth at penstock inlet

In this study, the Knauss formula has been considered for submergence depth value.

For design of the forebay total depth, different criteria have to be taken into account:

- The penstock invert elevation should be placed at least 0.2 m higher than the basin bed level.
- The minimum depth against vortex has to be accounted according to Knauss formula. This defines the minimum operation water level in the basin.
- For design of the forebay depth, a regulation reserve of 0.2 to 0.9 m (for discharges between 0.25 to 3.0 m^3/s) is considered above the minimum operation level of the forebay as a safe value based on experience [5].
- The difference between normal operation level of forebay and spillway level has been chosen as 0.1 m.
- The capacity of the lateral spillway is assumed two times the design discharge to keep a security margin. Hereby the maximum head over the spillway in operation is obtained.
- The basin has also a minimum additional freeboard of about 0.3 m over maximum water level regard to spillway operation.

Spillway dimension is determined based on the following formula:

$$Q_s = C_d \cdot B_s \cdot \sqrt{2 \cdot g} \cdot h_s^{3/2} \quad (4.9)$$

Q_s: design discharge of lateral spillway (m^3/s) C_d: discharge coefficient, 0.45

B_s: spillway width (m) h_s: design head over spillway (m)

4.4.8 Surge tank design criteria

A surge tank is more likely to be required when the headrace conduit is pressurized. Figure 4.32 illustrates the case of a junction of penstock with pressure conduit (point C) where a normal forebay is no longer convenient or possible. Effectively the adduction system composed of a pressure conduit of length L' and a penstock of length L, forms a long uninterrupted pressure conduit. On the occurrence of a water hammer; it is advisable to replace an enlarged forebay with a surge chamber and a vertical shaft of relatively large volume and sufficient height as shown on the Figure 4.32. The connection to the pressure conduit is made at the bottom N of the chamber. With this arrangement the continuity of the pressure system is separated in two lengths L and L'. As long as the plant is out of service, the water level in the surge tank is at the static level (point P) equal to the level of the intake.

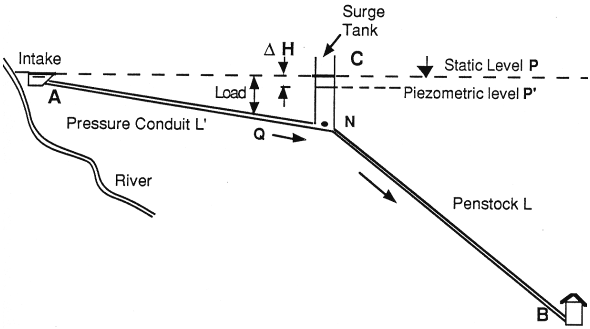


Figure 4.32: Description of a typical system with surge tank [5]

4.4.8.1 Water hammer

Water hammer is a term which refers to the transient pressure peaks which occur in a pipe when there is a rapid change in the flow velocity within it.

4.4.8.2 Description of phenomenon

Figure 4.33 illustrates how a velocity change caused by an instantaneous closure of a gate at the end of a pipe creates pressure waves traveling within the pipe [6].

Initially, water flows at some velocity "v_0" as shown in Fig. 4.33a. When the gate is closed, the flowing within the pipe has a tendency to continue flowing because of its momentum. Because it is physically prevented from so doing, it pipes up behind the gate; the kinetic energy of the element of water nearest the gate is converted to pressure energy, which slightly compresses the water and expands the circumference of the pipe at this point (Fig. 4.33b).

This action is repeated by the following elements of water (Fig. 4.33c), and the wave front of increased pressure travels the length of the pipe until the velocity of the water "v_0" is destroyed, the water is compressed, and the pipe is expanded its entire length (Fig. 4.33d).

At this point, the water's kinetic energy has all been converted to strain energy of the water (under increased compression) and strain energy of the pipe (under increased tension).

Because the water in the reservoir remains under normal static pressure but the water in the pipe is now under a higher pressure, the flow reverses and is forced back into the reservoir again with velocity "v_0" (Fig. 4.33e).

As the water under compression starts flowing back, the pressure in the pipe is reduced to normal static pressure. A pressure "unloading" wave then travels down the pipe toward the gate (Fig. 4.33f) until all the strain energy is converted back into kinetic energy (Fig. 4.33g).

However, unlike case (Fig. 4.33a), the water is now flowing in the opposite direction and because of its momentum; the water again tries to maintain this velocity. So, it stretches the element of water nearest the gate, reducing the pressure there and contracting the pipe (Fig. 4.33h).

This happens with successive elements of water and a negative pressure wave propagates back to the reservoir (Fig. 4.33i) until the entire pipe is under compression and water under reduced pressure (Fig. 4.33j).

The negative pressure wave would have the same absolute magnitude as the initial positive pressure wave if it is assumed that friction losses do not exist. The velocity then returns to zero but the lower pressure in the pipe compared to that in the reservoir, forces water to flow back into the pipe (Fig. 4.33k).

The pressure surge travels back toward the gate (Fig. 4.33e) until the entire cycle is complete and a second cycle starts (Fig. 4.33b). The velocity with which the pressure front moves is a function of the speed of sound in water modified by the elastic characteristics of the pipe material.

In reality, the penstock pipe is usually inclined but the effect remains the same, with the surge pressure at each point along the pipe adding to or subtracting from the static pressure at that point. Also, the damping effect of friction within the pipe causes the kinetic energy of the flow to dissipate gradually and the amplitude of the pressure oscillation to decrease with time.

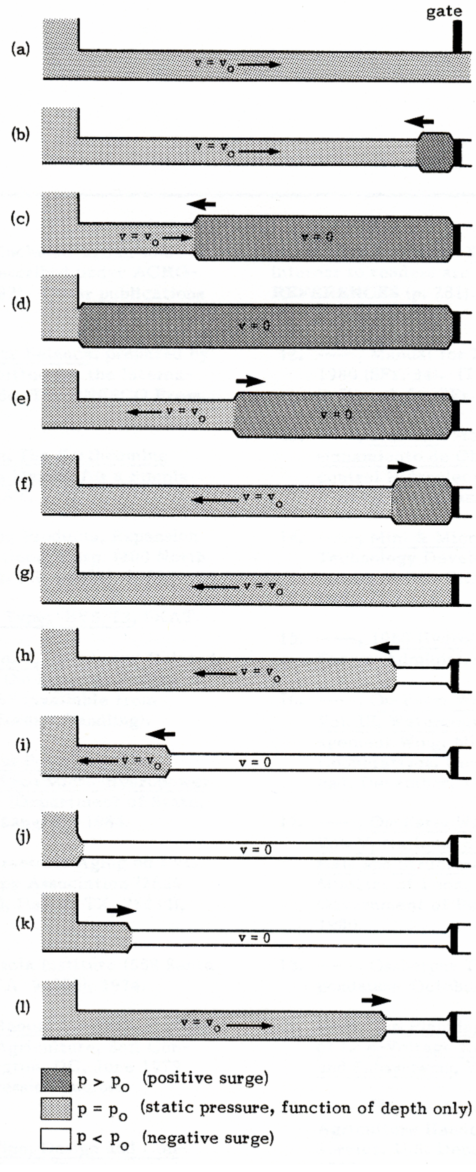


Figure 4.33: Movement along a pipe of a pressure surge caused by a sudden gate closure [6]

4.4.8.3 Critical time (reflection time)

Although some valves close instantaneously, closure usually takes at least several seconds. Still, if the valve is closed before the initial pressure surge returns to the gate end of the pipeline (Fig. 4.33g), the pressure peak will remain unchanged. All the kinetic energy contained in the water near the gate will eventually be converted to strain energy and result in the same peak pressure as if the gate were closed instantaneously. However, if the gate has been closed only partially by the time the initial pressure surge returns to the gate (Fig. 4.33g), not all the kinetic energy will have been converted to strain energy and the pressure peak will be lower. If the gate then continues closing, the positive pressure surge which it would then create will be reduced somewhat by the negative pressure (Fig. 4.33h) surge which originated when the gate originally began closing. Consequently, if the gate opens or close more time than required time for the pressure surge to travel to the reservoir and back to the gate, peak surge pressure are reduced. This time is called the critical time "T_c" and is equal to [6]:

$$T_c = \frac{2 \cdot L}{a_c} \tag{4.10}$$

a_c: wave velocity or speed of sound in water (m/s) L: pipeline length (m)

The wave velocity in a pipe is the speed with which the pressure surge travels along the pipe. It is a function of both the elastic characteristics of water and the pipe material. An expression for the wave velocity is [1]:

$$a_c = \frac{1}{\sqrt{\rho_w \cdot \left(\frac{D}{t \cdot E_T} + \frac{1}{E_w} \right)}} \tag{4.11}$$

t: wall thickness of penstock (m) E_T: module of elasticity of penstock ($2.1*10^{11}$ N/m^2)

ρ_w: density of water (kg/m^3) E_w: module of elasticity of water ($2*10^9$ N/m^2)

If the conduit is completely rigid then the propagation velocity of surge would be about 1400 m/s [1].

4.4.8.4 Head loss in pipe

The friction head loss in pressure pipe is calculated by the Darcy-Weisbach equation:

$$\Delta H = f \cdot \frac{L}{D} \cdot \frac{V^2}{2 \cdot g} \tag{4.12}$$

L: pipe length (m) f: friction coefficient D: penstock diameter (m)

V: mean velocity in pipe (m/s) ΔH: head loss (m) R: hydraulic radius (m)

D is the diameter of a circular cross section conduit or the hydraulic diameter given by *D=4*R.* Friction coefficients for circular sections are presented in terms of Reynolds number, Re, and relative roughness, k_s/D. The friction coefficients are presented graphically by the Moody chart, which is a plot of the Colebrook-White equation that requires an iterative solution. An equation of similar accuracy to the Colebrook-White equation that allows the friction coefficient to be obtained directly is [16]:

$$f = \frac{0.25}{\left[\log\left(\frac{k_s}{3.7 \cdot D} + \frac{5.74}{\mathrm{Re}^{0.9}} \right) \right]^2} \tag{4.13}$$

k_s: average roughness height of irregularities on the pipe wall (0.0015m for steel)

Re: Reynolds number, $V \cdot D / \nu$ ν: kinematic viscosity of water ($1.316 * 10^{-6}$ m^2/s)

4.4.8.5 Maximum oscillations

The following formula can be used for first estimation of extreme oscillations in a simple surge tank considering the total head losses "ΔH" in a pressurized headrace pipe [2]:

- Maximum oscillation (positive) in case of sudden closure of turbine gate (Z_{max}):

$$Z_{\max} = V_0 \cdot \sqrt{\frac{L' \cdot A_0}{g \cdot A_s}} - 0.60 \cdot \Delta H \qquad (4.14)$$

- Minimum oscillation (negative) in case of sudden opening of turbine gate (Z_{min}):

$$Z_{\min} = -V_0 \cdot \sqrt{\frac{L' \cdot A_0}{g \cdot A_s}} - 0.25 \cdot \Delta H \qquad (4.15)$$

V_0: flow velocity in headrace pipe (m/s)
L': length of headrace pipe (m)
A_0: area of pressurized pipe (m^2)
ΔH: head loss in headrace pipe (m)
A_s: area of surge tank (m^2)

4.4.8.6 Surge tank area

The minimum surge tank area should be specified by taking into account the stability criteria of Thoma. According to this criterion, in order to be able to damp out the mass oscillation in the surge tank, the section should be greater than a certain value and increased by a safety factor of 1.5 to 2.0 as following formula [1]:

$$A_s > \frac{A_0 \cdot V^{*2}}{2 \cdot g \cdot (H - \Delta H)} \quad ; \qquad V^* = K \cdot R^{2/3} \qquad (4.16)$$

V*: reference velocity (m/s)
K: Strickler coefficient for headrace pipe ($m^{1/3}/s$)
R: hydraulic radius of headrace pipe (m)
H: total head (m)

4.4.9 Standardization charts for forebay and surge tank

Standardization of forebay or surge tank consists of volumetric functions for different elements of civil works and is appropriately chosen to provide for hydraulic, structural and stability design considerations. Design charts are presented in Figures 4.34 to 4.36 for a forebay basin.

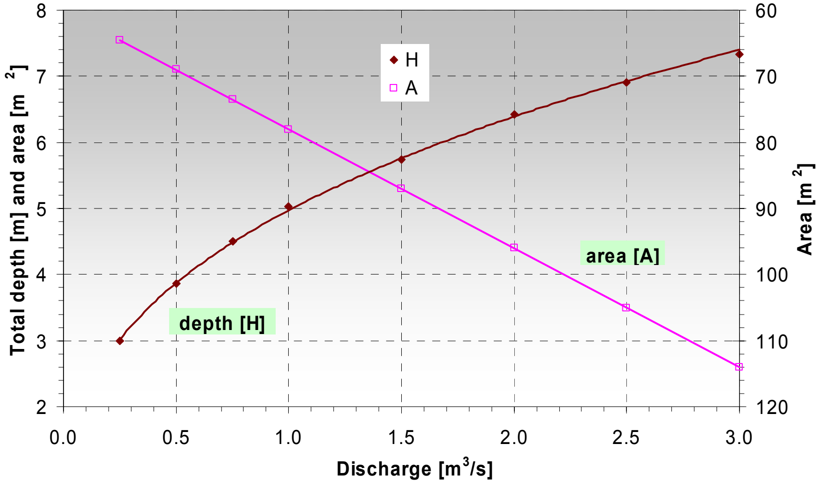


Figure 4.34: Design charts for forebay: required depth and plan area as a function of discharge

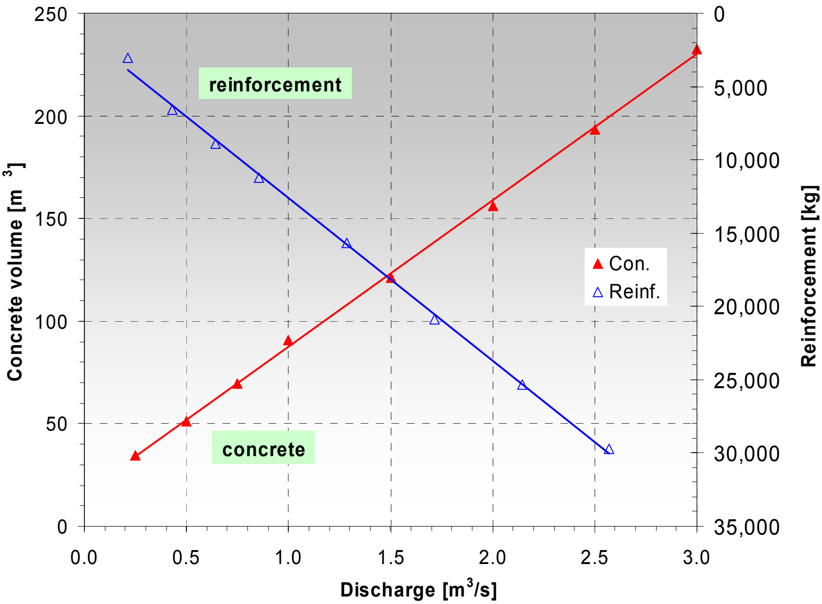


Figure 4.35: Design charts for forebay: required concrete volume and reinforcement as a function of discharge

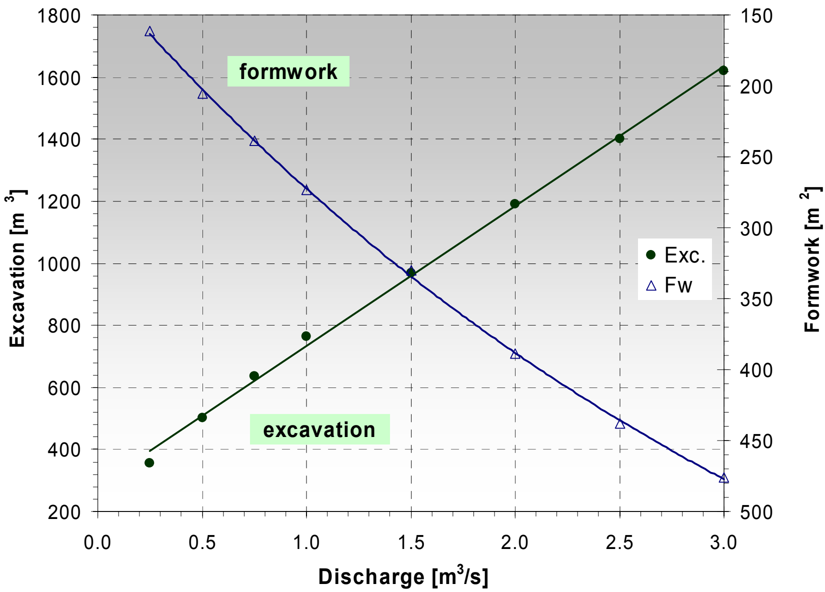


Figure 4.36: Design charts for forebay: required excavation and formwork as a function of discharge

Figures 4.37 to 4.41 shows the design charts for surge tank as a function of discharge and different length of the headrace system. According to the formula presented before about oscillations in a simple surge tank, the length of the pressure pipe is an important factor and has influence over surge tank dimensions. Therefore, for a series of pipe lengths and discharges, the standardization charts have been presented.

All of the standardization results of forebay and surge tank are presented in Appendix D.

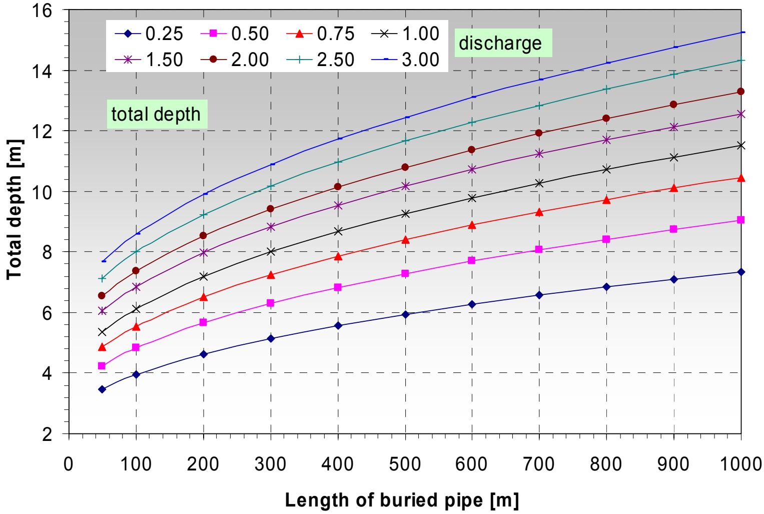


Figure 4.37: Design charts for a simple surge tank: required depth as a function of discharge (m^3/s) and headrace pipe length

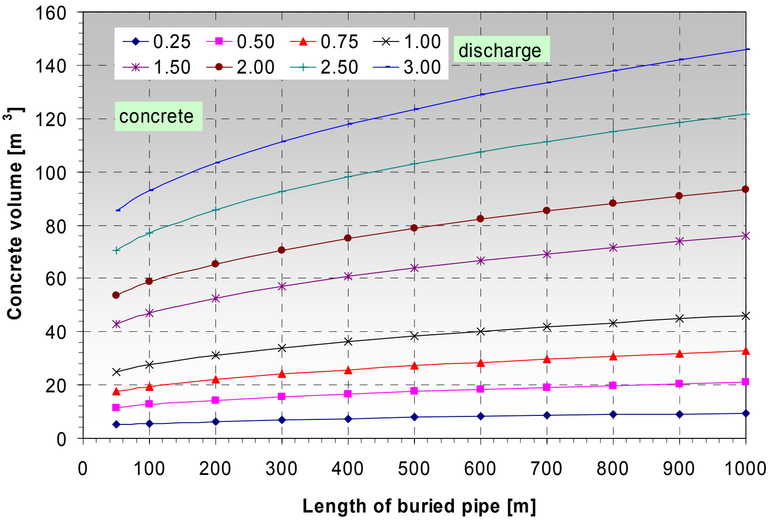


Figure 4.38: Design charts for a simple surge tank: required concrete volume as a function of discharge (m^3/s) and headrace pipe length

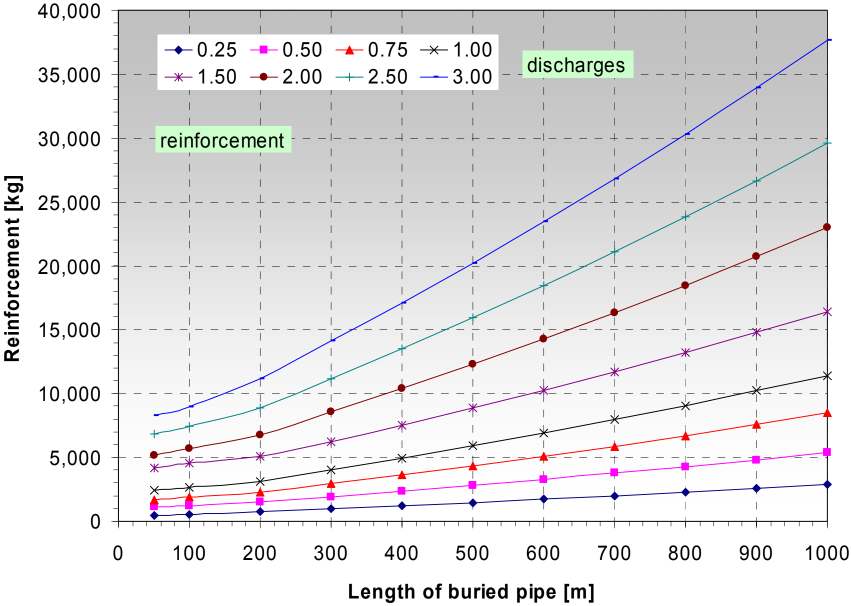


Figure 4.39: Design charts for a simple surge tank: required reinforcement as a function of discharge (m^3/s) and headrace pipe length

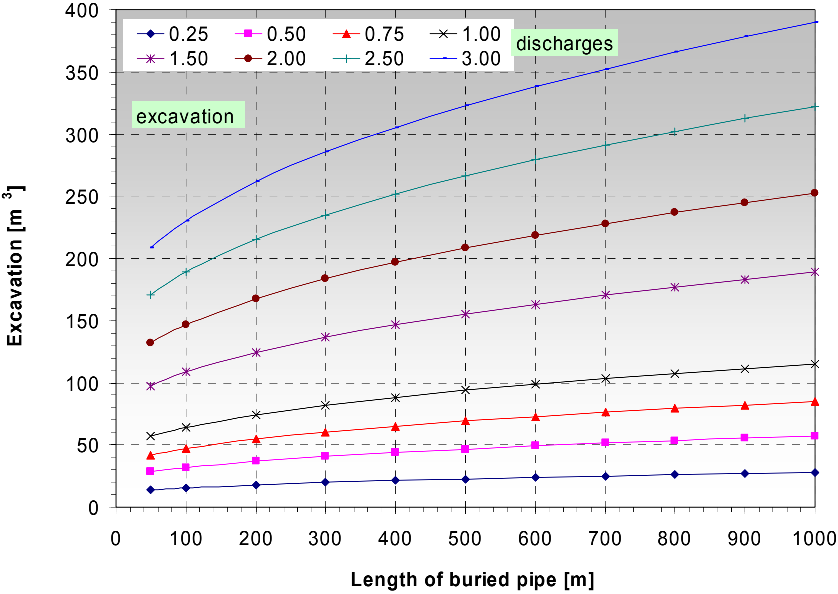


Figure 4.40: Design charts for a simple surge tank: required excavation as a function of discharge (m^3/s) and headrace pipe length

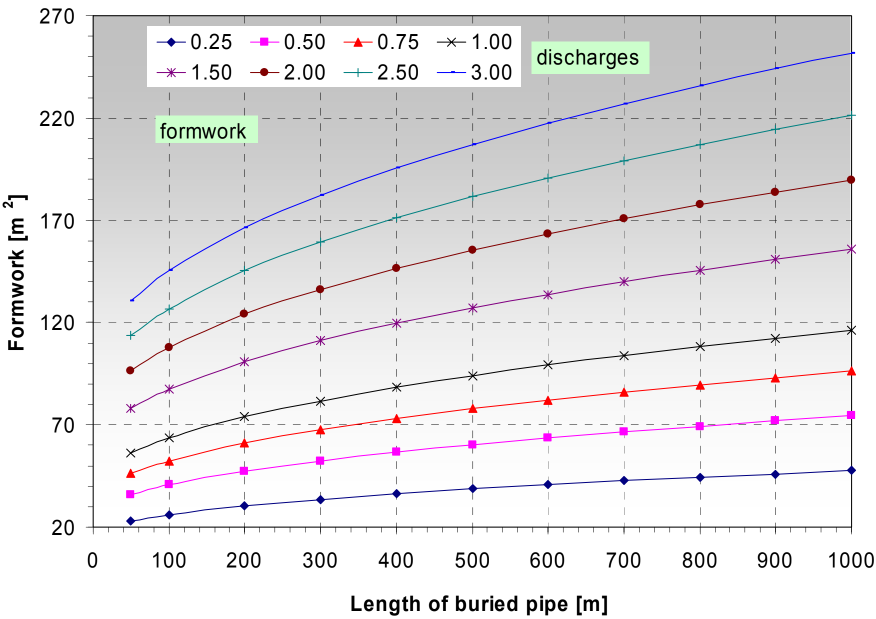


Figure 4.41: Design charts for a simple surge tank: required formwork as a function of discharge (m^3/s) and headrace pipe length

4.5 Penstock

The penstock is normally the most expensive part of a small hydropower scheme. It conveys the water via the shortest possible way from the forebay into the power house.

In selecting the type of penstock for a specific site, the following factors should be considered:

- required operating pressure and diameter
- method of coupling
- weight and ease in handling and accessibility of site
- local availability of pipe
- maintenance requirement and expected life
- nature of terrain to be traversed
- water quality, climate, soil and possible tampering

4.5.1 Sizing penstock pipes

When sizing a penstock pipe, two parameters must be specified:

- Its diameter, which should be selected to reduce friction and therefore energy losses within the penstock to an acceptable level.
- the thickness of its walls, which should be selected to accommodate the pressures encountered during plant operation.

4.5.2 Material

The most commonly used material for penstocks pipe is steel (Figure 4.42), however in some cases for low and medium head of schemes and short lengths, a simple and small PVC pipe can be used. In this last case, the PVC pipe should be placed under the ground due to degradation of pipe material against sunlight.

Figure 4.42: Examples of penstocks with support and anchor blocks

4.5.3 Arrangement

Penstocks for small hydropower are usually of the open air type, but sometimes, due to nature of the ground itself, the penstock material, the ambient temperatures and the environmental requirements, the buried type is preferred. The alignment of the steel conduit should be adapted to the topographical condition.

As mentioned before, a penstock pipe can be installed above or below ground. Flexible, small diameter penstocks used for plants with very low outputs are sometimes draped over the terrain down the hillside; however, large diameter penstocks installed above the ground must be secured properly to prevent movement which could damage the pipe. Although this increases the cost of the penstock construction and maintenance, the alternative of excavation for a buried pipe might be no less costly. The installation of a penstock above the ground increases its exposure to the elements, but its accessibility also facilitates inspection, maintenance, or repairs. Burying a penstock may involve considerable time and expense, but it protects the pipe from the elements and from the landslides, falling rocks and brush fires. In addition, the compacted soil firmly secures the penstock, providing adequate anchorage for most small diameter pipes.

4.5.4 Expansion joint

When the penstock exceeds a certain length, expansion joints are provided to take into account displacements in the longitudinal direction caused by temperature differences and Poisson's effect.

A penstock above ground is subject to greater temperature variations. If the turbine is not functioning continuously, these temperature variations can be pronounced, because the effect of flowing water with fairly constant temperature is not felt. Variations in temperature result in thermal expansion, which in turn causes stresses in the pipe. Thermal expansion and contraction are greatest for a penstock which is likely to remain empty during construction or repair, and provision must be made to accommodate these; either the pipe must be designed to have sufficient structural rigidity with the help of anchors and supports or expansion joints should be incorporated (Figure 4.43). These joints also help protect the pipe against earth tremors and are sometimes used to accommodate a slight misalignment of two pipe sections.

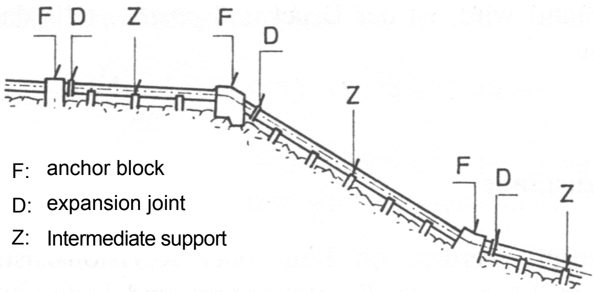


Figure 4.43: Typical description of different supports in a penstock [1]

4.5.5 Support

Rigid penstock pipes frequently require support piers, anchors, or thrust blocks to resist forces which can displace the pipe (Figure 4.43). Supports piers are used along straight runs of exposed pipe, primarily to prevent the pipe from sagging and becoming overstressed. They might also have to resist the longitudinal forces resulting from temperature induced movement of the pipe over the support. The pipe usually lays in a saddle on a reinforced concrete support piers and should be free to move longitudinally, to accommodate small pipe movements without abrading or cutting the pipe material (Figure 4.44). The spacing of the support piers is determined by a maximum unsupported span associated with the specific penstock pipe material and size. For buried pipe, the bed of soil on which it lies supports it, and separate supports are not generally required.

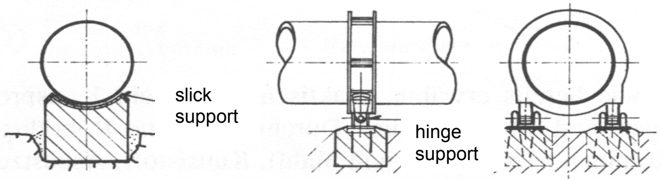


Figure 4.44: Different types of intermediate supports [1]

4.5.6 Anchor block

When there is a longitudinal slope change in the penstock, solid anchor blocks have to be provided to ensure that the resisting forces will not displace and damage the rigid pipe.

Significant forces can be concentrated at bends along a rigid, exposed penstock. The largest force is usually caused by the hydrostatic pressure within the pipe which tends to cause the penstock to crawl or the joints to separate. Depending on alignment and design of the penstock, other forces also contribute to a varying extent, such as those caused by thermal expansion of the pipe, the weight of upstream portion of pipe pushing downhill against the bend, and reductions in pipe diameters.

Anchors are incorporated at bends in the penstock either to provide the weight necessary to counteract the resultant of all these or simply to transmit it safely to the ground (figure 4.45). Even along a straight section of pipe down a steep slope, anchors may be required at intervals to prevent the pipe from sliding downhill because of its weight and they hold the pipe securely. An anchor block is usually constructed of concrete and held together and around the pipe with hoop reinforcement.

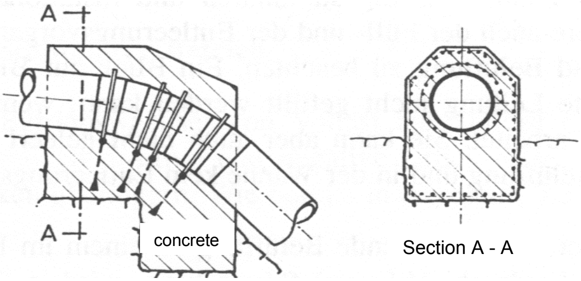


Figure 4.45: One typical solution for an anchor block of a penstock [1]

When the pipe is buried, properly compacted backfill generally serves the same function as anchors and support piers used with exposed pipe. Unless the penstock descends a steep slope, friction between the pipe and soil provides sufficient force to counteract the weight of the pipe pulling it downhill. Also, forces caused by thermally induced stresses are small because the pipe is shielded from large temperature variations.

4.5.7 Equipments

Gates or valves can be incorporated at either end of the penstock to control the flow of water. A turbine isolation valve is often included at the bottom of the penstock. It is used to turn the flow to the turbine on or off and is generally not used to regulate it. A valve supplied as part of the turbine generally provides this regulation, if required. Even thought this latter valve can also turn the flow to the turbine on or off, a shutoff valve is often included to perform this function if the flow regulating valve needs repair.

4.5.8 Design criteria for penstock

In the optimization process of penstock diameter, velocity and wall thickness, followings design criteria have been considered:

4.5.8.1 Diameter

The diameter of the penstock pipe is determined by the continuity equation:

$$D = \sqrt{\frac{4 \cdot Q}{\pi \cdot V}} \tag{4.17}$$

D: penstock diameter (m) Q: design discharge (m^3/s)

4.5.8.2 Maximum pressure

The maximum pressure in the penstock is estimated from the following equations [1]:

$$P_{max} = \rho_w \cdot g \cdot H + \rho_w \cdot a_c \cdot V \qquad \text{(sudden closure of turbine)} \qquad (4.18)$$

$$P_{max} = \rho_w \cdot g \cdot H + \frac{2 \cdot \rho_w \cdot V \cdot L}{T_c} \qquad \text{(progressive closure of turbine in time } T_c\text{)} \qquad (4.19)$$

H: total head between water level in forebay and tailrace channel of power house (m)

P_{max}: total maximum pressure in penstock (N/m^2)

V: flow velocity in pipe (m/s)

a_c: wave velocity in water (m/s)

ρ_w: density of water (1000 kg/m^3)

T_c: closure time of turbine (s)

L: length of penstock (m)

The first part of the above equation in related to static pressure and the second one to additional pressure created by the water hammer effect in the conduit. When the turbine gates are completely closed in an unexpected situation (in time zero), it is called the sudden closure which causes an instantaneous change in discharge.

This is an initial evaluation of the maximum pressure and the flexibility of the conduit and other influences should be taken into account in detail design.

4.5.8.3 Thickness

Normally the wall thickness of the penstock is determined conservatively by the formula 4.18 (sudden closure of turbine valves) as it gives higher value of maximum pressure and the second equation is just to control it. So the thickness of penstock wall is obtained according to following equation [1]:

$$t = \frac{P_{max} \cdot D}{2 \cdot \sigma_t} \qquad (4.20)$$

σ_t: admissible stress of steel (235 $*10^6$ N/m^2)

t: wall thickness of penstock (m)

4.5.8.4 Weight

The weight of penstock material should be determined for cost estimation as follows:

$$W_s = \frac{\pi}{4} \cdot (D'^2 - D^2) \cdot t \cdot L \cdot \rho_t \qquad (4.21)$$

D': external pipe diameter, D+2*t (m)

ρ_t: steel density (7850 kg/m^3)

W_s: total steel weight (kg)

4.5.8.5 Unit prices

During optimization of the penstock diameter, the unit cost of steel, including supports and joints, and also installation costs according to the penstock slope, has been defined at Table 4.7 [3].

Table 4.7: Reference unit costs for penstock optimization procedure in CHF

Steel weight*	Installation cost related to penstock slope [S]	
[CHF/ton]	S≤10%	S>10%
6000	(300+230*D)*L	(600+230*D)*L

* including all additional cost of intermediate supports and joints
L and D are in meter

4.5.8.6 Economical optimization

The factor for calculating the present value of an ordinary annuity may be calculated for amortization time period and interest rate. The annuity factor is the value of the following expression:

$$a_n = \frac{1}{i} \cdot \left[1 - \frac{1}{(1+i)^n} \right] \tag{4.22}$$

i: annual rate of interest (%)   n: amortization time period (year) a_n: annuity factor

The cash payment of "C_{an}" made at the end of each year for amortization time period "n" at annual interest rate "i" is given by:

$$C_{an} = \frac{C_T}{a_n} \tag{4.23}$$

C_{an}: annual cost (unit cost/year) C_T: Present value of total cost

These relations are used for optimization of the penstock diameter. The initial data are chosen according to values of Table 4.8.

Table 4.8: Reference initial data for economical analysis of penstock diameter

Time of amortization [n] [year]	Global efficiency [η] [%]	rate of interest [i] [%]
30	70	4.5

4.5.9 Design criteria for anchor block

By virtue of an anchor block weight and bearing area, it is designed to withstand any load the penstock may exert on it and anchor the pipe securely to the ground. The magnitude of force and its components act on an anchor block (Figure 4.46) due to change in momentum direction of flowing water in penstock are described below [1]:

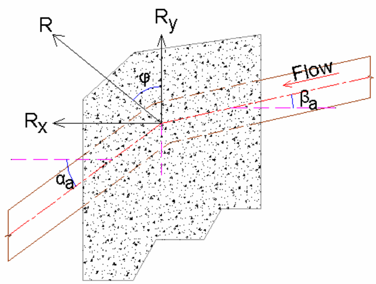


Figure 4.46: Flow acting force and its components on a typical anchor block

$$R_x = (\cos(\beta_a) - \cos(\alpha_a)) \cdot (\rho_w \cdot Q \cdot V + P_{\max} \cdot A) \qquad R = \sqrt{R_x^{\ 2} + R_y^{\ 2}}$$

$$R_y = (\sin(\alpha_a) - \sin(\beta_a)) \cdot (\rho_w \cdot Q \cdot V + P_{\max} \cdot A) \qquad \tan(\varphi) = \frac{R_x}{R_y} \tag{4.24}$$

β_α: angle of u/s part of penstock with horizontal (°)   A: area of penstock (m²)

α_α: angle of d/s part of penstock with horizontal (°)   φ: angle of "R" with horizontal (°)

R: resulting force acting on anchor block (N) R_x , R_y: components of acting force (N)

The safety factors of overturning and especially sliding against this force determine the anchor block dimensions. The resisting force of bearing area is related to friction angle of foundation material "ϕ" and resulting forces in vertical directions.

There are also some other forces which act on anchor block as [1]:

- The conduit weight
- The water weight
- The longitudinal forces due to local change in diameter (if exist)
- The temperature effect (shortening or lengthening of the conduit if it is not provided with expansion joints)
- Shear forces on the conduit wall because of flow friction

4.5.10 Standardization charts for penstock

The most important part in design of a penstock is to select an optimum diameter having the minimum total cost over the technical life of the plant i.e. time of amortization.

First of all for each diameter, annual energy losses are calculated and plotted based on water loss in penstock and annual volume of water used for energy production.

For each diameter, the total annual cost of penstock is also computed and plotted. These computations should include all factors that might significantly influence its cost. Once both curves are plotted, they are added graphically. The optimum diameter is the one associated with the minimum total annual cost. Figure 4.47 show an example of these processes for a certain head and discharge.

It should be mentioned that in the present study different combinations of total head, penstock diameter, energy sale price and a series of discharges have been considered to cover all possible conditions in the optimization procedure (Table 4.9).

Table 4.9: Different combinations for penstock optimization procedure according to energy sale price, design discharge and head (320 conditions)

Energy sale price [CHF/kWh]	Design discharge [m^3/s]	Total head between forebay water level and tailrace canal of turbine [m]
0.04	0.25, 0.50, 0.75, 1.0, 1.5, 2.0, 2.5, 3.0	100, 200, 300, 400, 500, 600, 700, 800, 900, 1000
0.08	"	"
0.12	"	"
0.16	"	"

Figure 4.48 shows some of the optimization results for four series of energy sale price, total head as 200 m and different discharges. In Figure 4.49 the influence of total head over the optimum diameter is investigated and compared graphically for different energy sale prices. It is observed that the optimum diameter increases with energy sale price for constant head or discharge (Figure 4.48, 4.49). Figure 4.50 also shows the variation of optimum velocity with discharge and energy sale price for a total head of 200 m.

<u>All of the standardization results of penstock and anchor block are presented in Appendix E.</u>

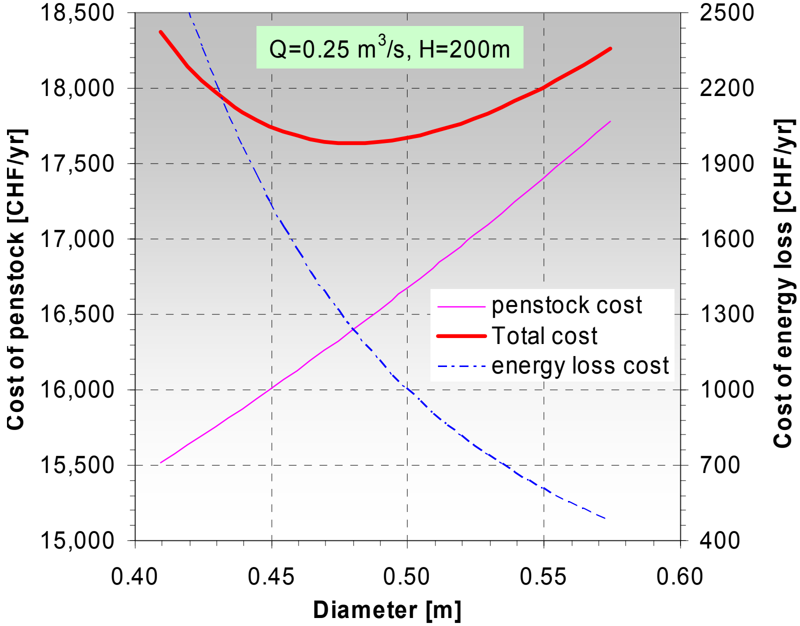


Figure 4.47: Optimization of penstock diameter for 0.16 CHF/kWh as energy sale price

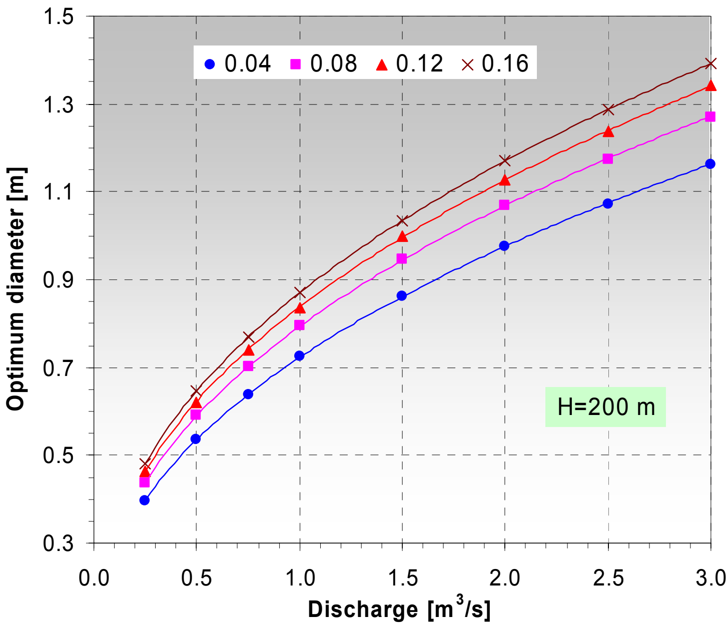


Figure 4.48: Optimization of penstock diameter for different energy sale prices

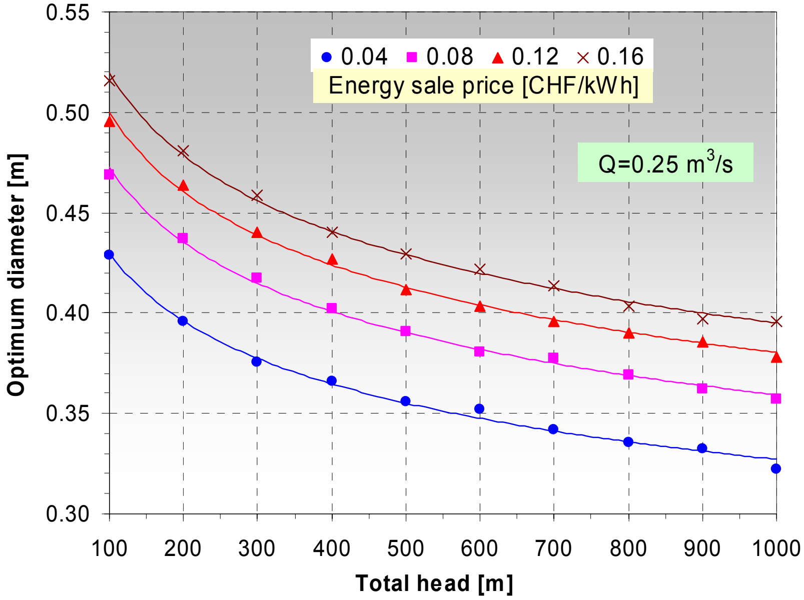


Figure 4.49: Optimization of penstock diameter for different total heads and energy sale prices

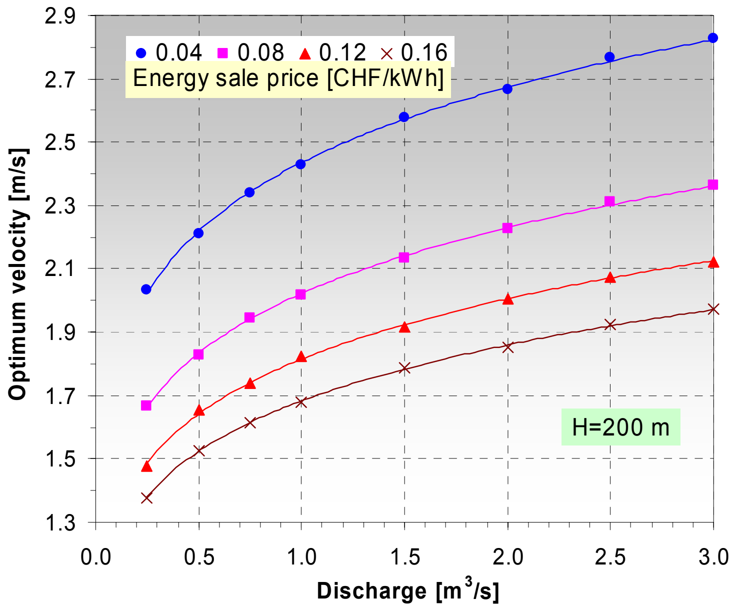


Figure 4.50: Optimum penstock velocity for different energy sale prices

4.5.11 Standardization charts for anchor block

In the standardization process the volumetric curves for anchor blocks are also obtained for three different deflection angles of 120°, 140°, 160° and for different discharges and total heads. Figures 4.51 and 4.52 show an example of the standardization charts for anchor block with deflection angle of 160° (α_a=70°, β_a=30°, Figure 4.46) and 100 m of head. It is clear that the cost of anchor blocks should be added to the total cost of penstock.

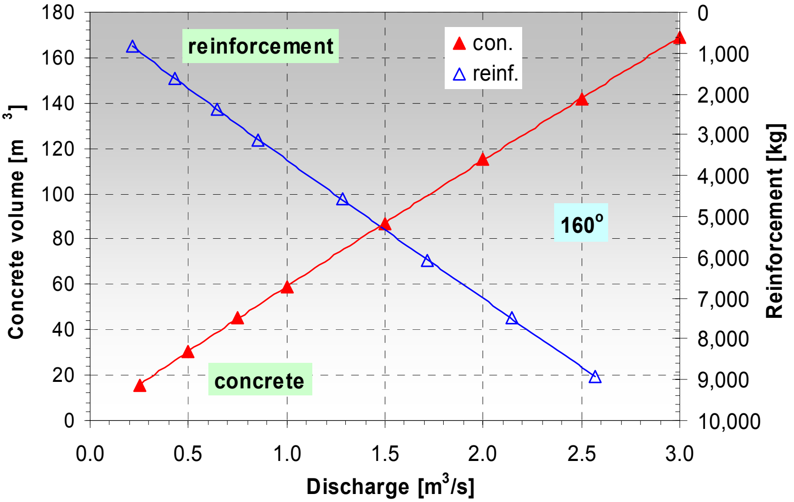


Figure 4.51: Design charts for anchor block with a deflection angle of 160° (α_a=70°, β_a=30°) and head of 100 m: required concrete volume and reinforcement as a function of discharge

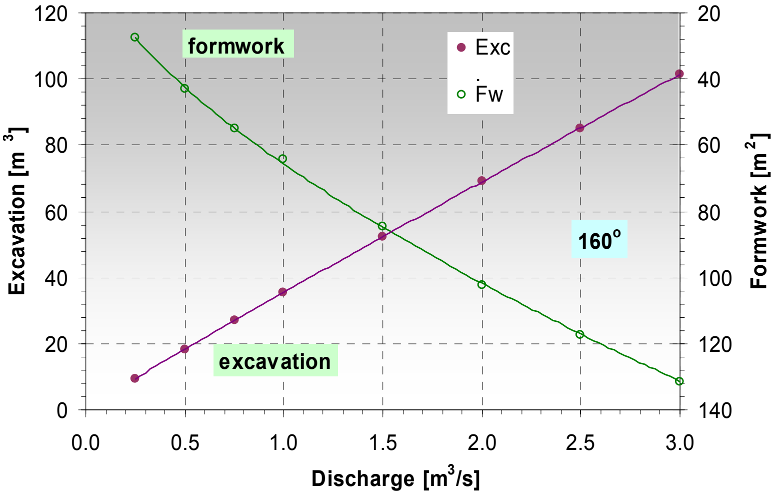


Figure 4.52: Design charts for anchor block with a deflection angle of 160° (α_a=70°, β_a=30°) and head of 100 m: required excavation and formwork as a function of discharge

5 Optimization tool 'POPEHYE'

After standardization of all components of small hydro based on civil works, the final volumetric curves and cost functions are implemented in an existing optimization tool called "POPEHYE". This software was developed within the framework of a collaboration between "l'Ecole d'Ingénieurs d'Yverdon" (EIVD) and the "l'Ecole Polytechnique Fédérale de Lausanne" (EPFL) at the Laboratory of Hydraulic Constructions (LCH), Switzerland. The major part of this program has been developed by Gianluca Gatti with the scientific support of Dr Jean-Louis Boillat.

The program allows a step by step design and optimization tool for primary evaluation of different alternatives for small hydropower plants, according to location, head, discharge and power production. Three different steps in POPEHYE program are recognized (Figure 5.1):

- Hydrological analysis of the catchment area
- Economical optimization of the design discharge
- Pre-dimensioning of the main hydraulic structures

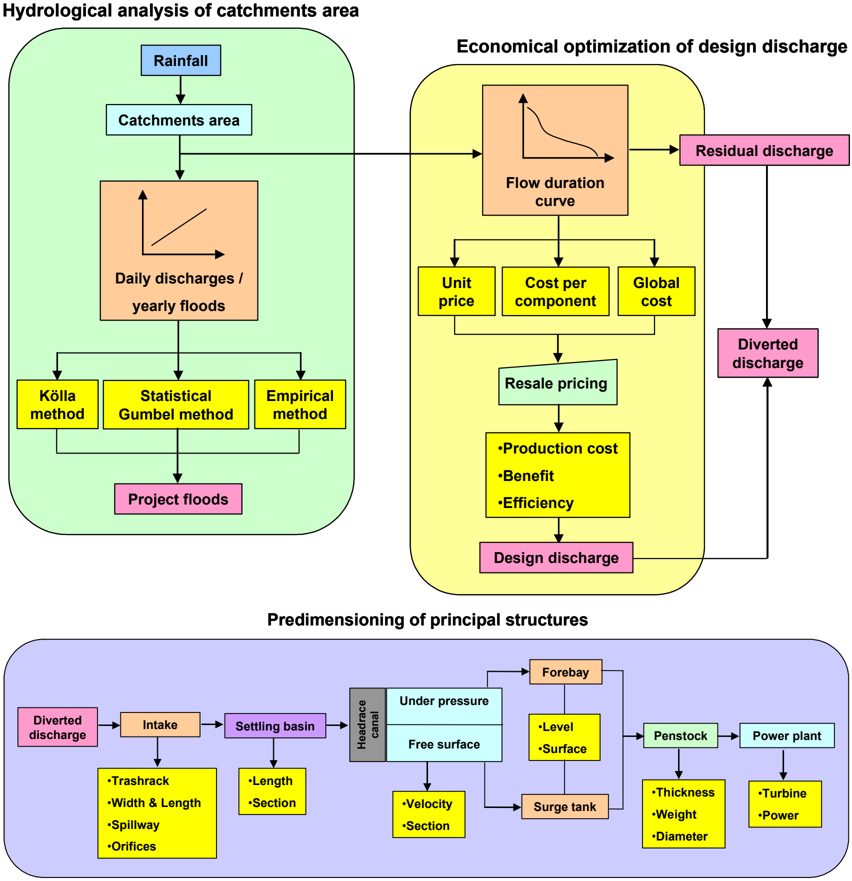


Figure 5.1: Flow charts of major steps of the optimization tool "POPEHYE"

5.1 Hydrological analysis of the catchment area

For the use of the program "POPEHYE" it is important to have some preliminary data about the catchment area. This hydrological data is necessary for estimation of flow duration curve and then design discharge for power plant. The main objective is to compute the flood with a 100 year return period (HQ_{100}), to control the hydraulic capacity of river intake and spillway. The flood discharge can be obtained by using different statistical and empirical means as ‘‘Koella method’’, ‘‘statistical method of Gumbel’’ and ‘‘empirical methods’’.

Koella’s method is based on knowledge of the catchment area but, based on experience, it results in rather low values for the 100 year flood. The statistical method of Gumbel adjustment is based on analysis over a series of annual maximum discharges measured in the catchment area. This method gives the most proper values for flood discharge as far as it is based on frequency analysis over measured discharges. Empirical methods provide ranges of extreme values for the flood discharge. The final value of the 100 year flood of a catchment area is generally obtained by analysis of results between different methods.

5.1.1 Koella’s method

This method makes it possible to determine first 20 year flood (HQ_{20}) when there are no direct measurements of river discharge at the site. It is based on characteristic of the catchment area such as topography, geology and precipitations and has been developed for conditions in Switzerland. The flood discharge of 100 year return period (HQ_{100}) will be obtained by extrapolation and correction factor over HQ_{20}. The principal steps of Koella’s method are shown in Figure 5.2:

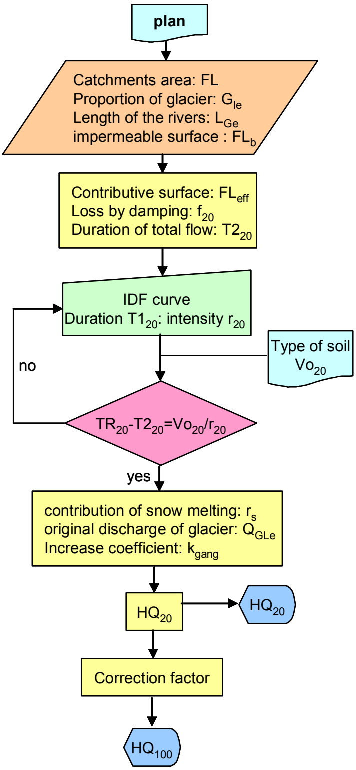


Figure 5.2: Flow charts of major steps in Koella method [3]

5.1.1.1 Catchment area

As first step the surface area of watershed (FL, km^2) and proportion of glacier part (G_{le}) are obtained according to the available map and plan of the site.

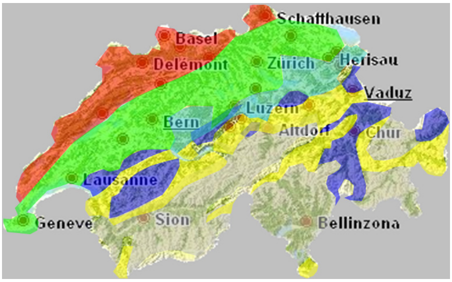
5.1.1.2 Contributive effective surface

In this step the cumulative length of all river branches in the watershed (L_{Ge}, km) are computed. Then the contributive effective surface (FL_{effX}, Km^2) will be obtained by:

$$FL_{effX} = 0.12 \cdot L_{Ge}^{\;1.07} \tag{5.1}$$

5.1.1.3 Soil classification

Classifications of materials in the project area are obtained by the map of different kind of soils and rocks like Figure 5.3 for Switzerland.



red	limestone of the Jura
green	moraine
yellow	alpine limestone
blue	sandstone
gray	granite
bright blue	consolidated moraine

Figure 5.3: Soil classification map of Switzerland in Koella's method

5.1.1.4 Wet volume and loss

Wet volume for the 20 year flood (Vo_{20}, mm) will be evaluated according to the soil classification presented in Table 5.1.

Table 5.1: Criteria for determination of wet volume Vo_{20} for different kinds of soils [3]

Landschafts-charakteristik	Alpine kristalline Berg-landschaften		Alpine Kalk-Berg-landschaften		Flysch-/ Bündner-schiefer-landschaften		Voralpine Nagelfluh-landschaften		Molasse-landschaften des Mittellandes		Jura-Kalk-landschaften	
	Standard	Trogtäler mit Talschotter	Standard	Trogtäler mit Talschotter	Standard	Trogtäler mit Talschotter	Standard	Sohlentäler	Standard	Sohlentäler	Standard	Sohlentäler
Bodenkarte	V,W	R,Y (V8,W8)	U	R,Y (U8)	S (T)	R,Y (Q)	M,N,O, P,X,(G)	R,Y (Q)	G,H, K,L	G1,H4, J1,Q	A,B,C, D,E	B2,B5
BASISWERT	E	B	C	B	D	B	C	B	D	B	C	B
Tiefgründigere Böden (starke Bewaldung o.ä.)	F	C	D	C	D	C	C	C	D	C	D	C
Flachgründigere Böden (oberflächennahe Stauschichten o.ä.)	D	B	B	B	B	B	B	B	D	B	C	B
Starkes Relief	E	B	C	B	C	B	B	B	C	B	B	B
Schwaches Relief	E	B	C	B	D	B	D	B	E	B	C	B
FL_{eff} hauptsächlich in der Nähe des Vorfluters	B	A	B	A	B	A	C	B	D	A	C	A

A	20*
B	25
C	30
D	35
E	40
F	45

* all the values in mm

Then, the infiltration loss f_{20} can be evaluated as:

$$f_{20} = 0.1 \cdot Vo_{20} \text{ (mm/h)} \tag{5.2}$$

5.1.1.5 Duration and intensity of rain

The rainfall specifications are simulated with following formulas:

$$TR_{20} = T1_{20} + T2_{20} \qquad T2_{20} = FL_{eff\,20}^{\;0.2} \qquad (5.3)$$

$T1_{20}$: wet duration of 20 years flood (h) $T2_{20}$: Duration of total flow of 20 years flood (h)

TR_{20}: Precipitation time for 20 years flood (h)

By iteration over TR_{20} and below formula and also with the help of IDF curve (intensity-duration-frequency) the correct values of Vo_{20} will be reached.

$$r_{20} \cdot T1_{20} = Vo_{20} \qquad (5.4)$$

r_{20}: precipitation intensity of duration TR_{20} for 20Y flood (mm/h) IDF curve: $r_{20}=f\,(TR_{20})$

5.1.1.6 Flood discharge (HQ_{20})

The flood discharge for a 20 year return period is then calculated based on the formula below:

$$HQ_{20} = (r_{20} + r_s - f_{20}) \cdot (FL_{eff\,20} + FL_b) \cdot k_{Gang} \cdot \frac{1}{3.6} + Q_{Gle20} \ \ (\mathrm{m^3/s}) \qquad (5.5)$$

r_s=0.4 contribution of snow melting (mm/h) FL_b: impermeable surface (km^2)

$$Q_{GLe} = (0.35 to 0.50) \cdot GLE_{AN} \cdot FL \qquad (5.6)$$

GLE_{AN}: proportion of glacier surface of watershed Q_{GLe}: original discharge of glacier (m^3/s)

For catchment area less than 10 km^2 and rainfall durations lower than 3h, the value for K_{Gang} is:

$$TR_X \le 1h: \qquad k_{Gang} = 1 + \frac{10 - FL}{9} \cdot 0.2 \qquad FL \ge 1km^2 \quad ; \quad k_{Gang} = 1.2 \qquad FL < 1km^2$$

$$1h \le TR_X \le 3h: \qquad k_{Gang} = 1 + \frac{3 - TR_X}{2} \cdot \frac{10 - FL}{9} \cdot 0.2 \qquad FL \ge 1km^2$$

$$k_{Gang} = 1 + \frac{3 - TR_X}{2} \cdot 0.2 \qquad FL < 1km^2 \qquad (5.7)$$

5.1.1.7 Extrapolations

The last step is to compute the 100 year flood discharge considering the coefficients of Table 5.2.

Table 5.2: Coefficients of volume and surface for extrapolation

Category	Vo_{20} [mm]	f_{20} [mm/h]	Vo_{100} [mm]	f_{100} [mm/h]	K_{F100}
A	20	$f_{20}=0.1Vo_{20}$	$Vo_{100}=1.3Vo_{20}$	$f_{100}=0.1Vo_{100}$	1.10
B	25				1.15
C	30				1.20
D	35				1.25
E	40				1.30
F	45				1.30

The coefficient K_{F100} is substituted in the formula below:

$$FL_{eff\,100} = K_{F100} \cdot FL_{eff\,20} \qquad (5.8)$$

5.1.2 Statistical method of Gumbel

Gumbel distribution is one of the most widely used probability analysis for extreme values in hydrological studies for prediction of floods with different return periods. Statistical method of Gumbel proposes a double exponential distribution to adjust a series of maximum annual discharges. This method consists of the following steps (Figure 5.4):

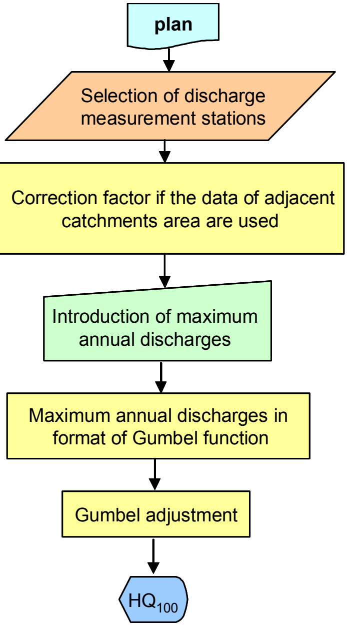


Figure 5.4: Flow charts of major steps in Gumbel method [3]

5.1.2.1 The frequency distribution

Frequency distribution function is [17]:

$$F(Q) = \exp[-\exp(-u)] \quad ; \quad u = \frac{Q - a_g}{b_g} \tag{5.9}$$

The parameters are estimated as given below:

$$a_g = \overline{Q} - 0.5772 \cdot b_g \quad ; \quad b_g = \frac{\sqrt{6} \cdot S}{\pi} \quad ; \quad F(Q) = \frac{T-1}{T} \tag{5.10}$$

Substituting the values in Gumbel function yields to:

$$u = -\ln\left[\ln\left(\frac{T}{T-1}\right)\right] \tag{5.11}$$

a_g: mode of distribution u: reduced variable T. return period

$\overline{Q}$: average value of maximum annual discharges S: standard deviation of discharge values

The data of maximum annual discharges should be sorted in ascending order and then Hazen function is exerted to each row of data for computation of probability and reduced variable [17]:

$$P(Q \geq Q_m) = \frac{m - 0.5}{n_g} = \frac{1}{T} \quad ; \quad u_m = -\ln\left[\ln\left(\frac{T}{T-1}\right)\right] \tag{5.12}$$

m: number of each row of discharges n_g: total number of discharge values

P: probability function related to return period u_m: reduced variable for each row

According to these stages it is possible to estimate the flood value of desired return period (HQ_{100}) based on measured discharges for a series of statistical years.

5.1.3 Empirical methods

The empirical equations give an evaluation of the extreme discharge and are based on the measured maximum specific discharges (q_{max}, $m^3/s.km^2$). The available formulas are *Kürsteiner*, *Hofbauer*, *Melli*, *Müller* and *Lauterburg* which have been developed for catchment areas in Switzerland and should be adapted for other conditions. These methods make use of certain characteristics of the catchment area (Figure 5.5).

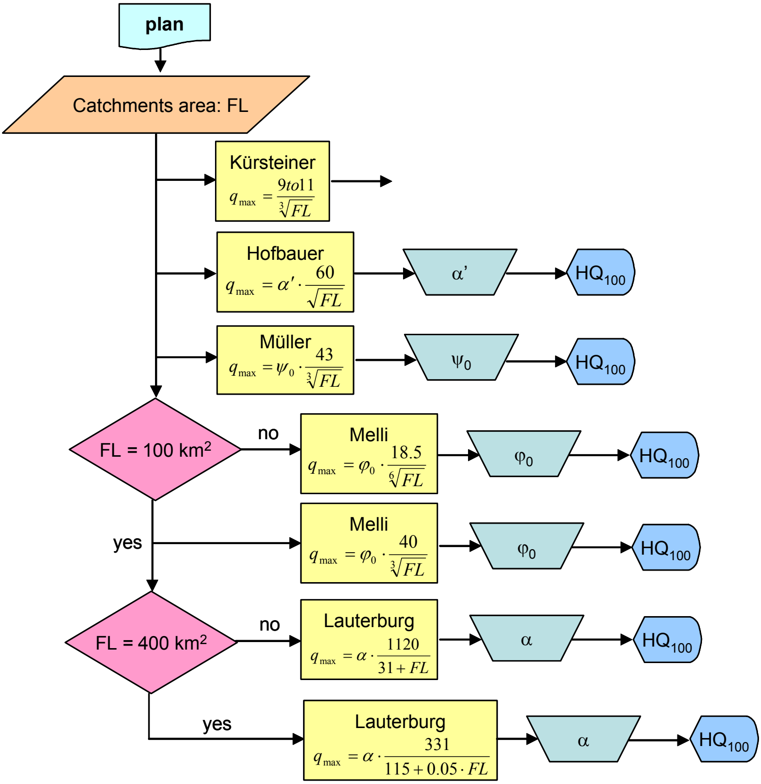


Figure 5.5: Flow charts of different empirical formula used in alpine catchment areas (Switzerland)

5.1.3.1 Kürsteiner equation

The maximum specific discharge is (m^3/s per km^2):

$$q_{\max} = \frac{9to11}{\sqrt[3]{FL}} \qquad \text{FL: surface of catchment area (km}^2\text{)} \qquad (5.13)$$

5.1.3.2 Hofbauer equation

The maximum specific discharge is as below:

$$q_{\max} = \alpha' \cdot \frac{60}{\sqrt{FL}} \qquad (5.14)$$

Parameter α' depends on the characteristics of the site and is obtained from Table 5.3:

Table 5.3: Coefficient of α' for Hofbauer equation

α'	Site characteristics
0.25 - 0.35	flat
0.35 - 0.50	Hills or mountains with mean height
0.50 - 0.70	Mountains

5.1.3.3 Melli equation

The maximum specific discharge for two range of catchment area is:

$$q_{\max} = \varphi_0 \cdot \frac{18.5}{\sqrt[6]{FL}} \qquad 0.3 \leq FL \leq 100 \text{ km}^2 \qquad (5.15)$$

$$q_{\max} = \varphi_0 \cdot \frac{40}{\sqrt[3]{FL}} \qquad \text{FL> 100 km}^2 \qquad (5.16)$$

Coefficient of φ_0 has an average value of 0.4.

5.1.3.4 Müller equation

The maximum specific discharge is:

$$q_{\max} = \psi_0 \cdot \frac{43}{\sqrt[3]{FL}} \qquad (5.17)$$

Coefficient of ψ_0 depends on the geology and land type which will be obtained from Table 5.4:

Table 5.4: Coefficient of ψ_0 for Müller equation

Area	Characteristics	Slope		
		Flat	Average	Steep
Above the limit of the forest	rock	0.4	0.6	0.8
Area limited with the forest	thick forest	0.3	0.5	0.7
	sparse forest	0.2	0.4	0.6
	farm	0.1	0.3	0.5
Low altitude	ordinary forest	0.1	0.2	0.4
	old forest	0.05	0.15	0.3

5.1.3.5 Lauterburg equation

The maximum specific discharge for two range of catchment area is:

$$q_{\max} = \alpha_L \cdot \frac{1120}{31 + FL} \qquad 1 \leq FL \leq 400 \text{ km}^2 \qquad (5.18)$$

$$q_{\max} = \alpha_L \cdot \frac{331}{115 + 0.05 \cdot FL} \qquad \text{FL> 400 km}^2 \qquad (5.19)$$

Coefficient of α_L depends on the slope, geology and vegetation of land area and can be obtained from Table 5.5:

Table 5.5: Coefficient of α_L for Lauterburg equation

Slope	α_1*	Geology condition	α_2*	Vegetation condition	α_2*
Very steep	0.3	Impermeable	0.3	stripped rock ground	0.30
Average	0.2	Average permable	0.2	Meadow and pasturage ground	0.25
Flat	0.1	Very permeable	0.1	cultivated ground or light timbering	0.15
*: $\alpha_L=\alpha_1 + \alpha_2 + \alpha_3$				Thick forest and high water absorbtion	0.05

5.2 Economical optimization of the design discharge

Hydrological analysis of a catchment area results in the flow duration curve. The optimization of discharge in the program “POPEHYE” is performed for all of the 365 daily discharges according to the FDC. Then the production cost, benefit and efficiency functions for all discharges are graphically presented. According to these last tree curves, the designer can choose the most economic and optimum design value for discharge in a SHP for a certain Head.

The present version of the software is available in French and called as POPEHYE Ver2, but the standardization curves and cost functions are not yet introduced in the optimization process. Three methods for cost evaluation of plant were available in Ver2 of the program (Figure 5.6).

Method 1

Unit costs

	discharge [m³/s]	cost [CHF]
Intake:	0.05	168,000
	0.50	230,000
	0.75	250,000
Headrace canal:	0.05	336
[Cost/m]	0.50	400
	0.75	450
Settling basin:	Cost/m²	5,000
Penstock:	Cost/ton	6,000
Powerhouse:	Cost/kW	800
Access road:	Cost/m	200

Method 2

Cost per components [CHF]

Water intake	210,000
Settling basin	135,000
Canal/Galery	100,000
Forebay/Surge tank	120,000
Penstock	820,000
Anchor blocks	85,000
Power house	900,000
Acces roads	30,000
TOTAL	**2,400,000**
Annuity	147,340

Method 3

Global cost

The application domain of this last method is limited because it was established on the analysis of schemes whose power is lower than 1MW.

Figure 5.6: Different methods introduced in POPEHYE Ver2 for optimization process

5.2.1 Unit cost method

In this method the unit prices of different components were introduced as a function of intake discharge, Headrace canal length, Settling basin area, penstock weight, power plant generation and access road length. This method was used in POPEHYE Ver2 for optimization of design discharge according to flow duration curve.

5.2.2 Total cost method

Cost functions in this method were not related to some design values and the total cost of each civil works was added by the user.

5.2.3 Global cost method

This method was applicable for plants with power less than 1 MW. The global cost was obtained by introducing a cost value for each kW of power production.

As it is observed from Figure 5.6, the cost functions which are the basic values for optimization process are estimated roughly and it is necessary to apply the new standardized functions in the POPEHYE.

5.2.4 Standardization charts in POPEHYE

In this study this program was upgraded in English and its accuracy in optimization strategy was increased by integrating the result of the standardization process for the main components of a small hydropower plant. It also follows the hydraulic design of different components of small hydro and performs an optimization of design discharge according to flow duration curves of the river. The new version of program is called “POPEHYE Ver2.2” with all standardization results and cost functions.

5.2.4.1 Water intake

The standardization charts of water intake depend on the parameters presented in Table 5.6. As an example, the standard table for drop intake with β=30°, a/b=1/3 and river bed in rock are shown in Figure 5.7. The volumetric design values are the first results in the program which are used furthermore for evaluation of cost function.

Table 5.6: Required parameters for intake structures in optimization process (POPEHYE Ver2.2)

Design discharge	River width	River bed type	Trashrack spacing	Slope angle of trashrack
Q [m³/s]	W [m]	Rock or Alluvium	a/b [1/3, 1/2, 2/3]	β [30°, 35°, 40°, 45°]

It should be mentioned that the appropriate unit costs of different civil works and incremental coefficients have to be introduced by the user of the program (Figure 5.7). In this case the last result of computations will be the corresponding cost function based on design discharges which is created automatically according to basic data of Table 5.6.

Unit costs

Concrete volume [m³]	Steel bar weight [kg]	Formwork surface [m²]	Excavation in Rock [m³]	Excavation in Alluvium [m³]	Backfilling [m³]
250	2.5	50	100	30	50
CHF	CHF	CHF	CHF	CHF	CHF

Project study [%]	Site installation [%]
10	30

Unit: CHF

1. Beta=30, a/b=1/3, rock

Q_{design}= 0.45 m³/s

w= 7.0 m

1.1. Volume of civil works

Concrete volume	Steel bar	Formwork	Excavation
m³	kg	m²	m³
46	**7682**	**127**	**59**

1.2. Cost of civil works

Concrete volume	Steel bar	Formwork	Excavation	Total cost
CHF	CHF	CHF	CHF	CHF
11,594	**19,204**	**6,373**	**5,921**	**61,623**

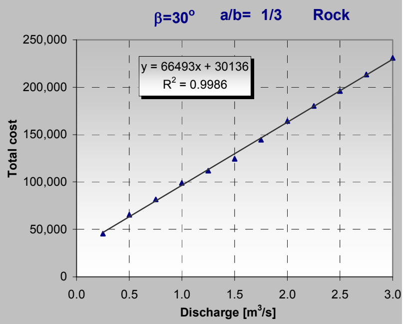


Figure 5.7: Standard format for intake in optimization process and cost function including unit prices of civil works (POPEHYE Ver2.2)

5.2.4.2 Settling basin

The standardization charts of settling basin depend on the parameters presented in Table 5.7. As an example, the standard table for Bieri settling basin in rocky bed with d=0.3mm are shown in Figure 5.8. The volumetric design values are the first results in the program which are used for estimation of cost function.

Table 5.7: Required parameters for settling intake in optimization process (POPEHYE Ver2.2)

Design discharge	Design grain size	Basin flushing type	Bed type
Q [m³/s]	d [0.2, 0.3 mm]	Bieri or Büchi	Rock or Alluvium

According to the unit costs, the last result will be the corresponding cost function based on design discharges (Figure 5.8).

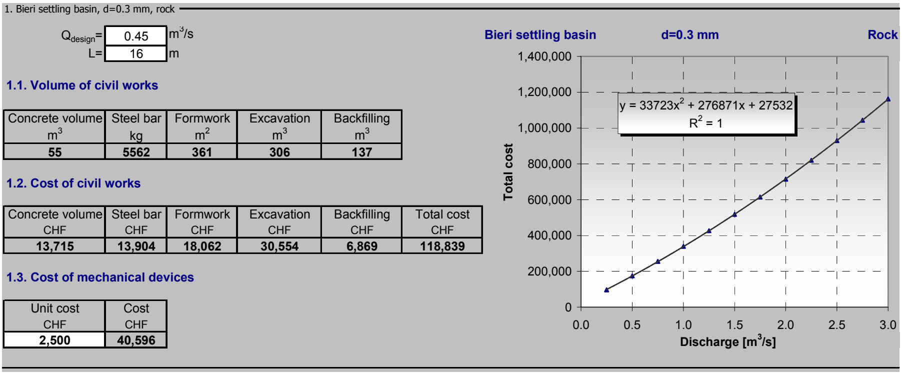


Figure 5.8: Standard format for settling basin in optimization process and cost function (POPEHYE Ver2.2)

5.2.4.3 Headrace canal

The standardization charts of headrace canal depend on the parameters presented in Table 5.8 as well as selected alternative. The standard table is presented in Figure 5.9 for an open concrete canal with a bed slope of 0.2 %, rectangular section and Length of 200m. The volumetric design values are used for evaluation of cost function.

Table 5.8: Required parameters for headrace canal in optimization process (POPEHYE Ver2.2)

Design discharge	Bed type	Canal length	Profile	Construction material	Flow regime	Canal slope
Q [m^3/s]	Rock	L' [m]	1. Open canal	concrete	Free surface	0.1 to 0.4 %
			2. Buried canal	concrete		
			3. Open canal	rock		
			4. Buried pipe	concrete		
			5. Buried pipe	PVC		
			6. Buried pipe	concrete	Under pressure	-

According to the unit costs, the corresponding cost function based on design discharges will be obtained as Figure 5.9.

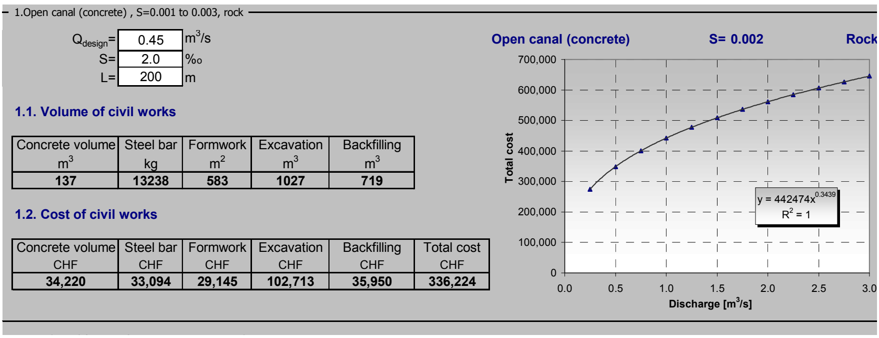


Figure 5.9: Standard format for headrace canal in optimization process and cost function (POPEHYE Ver2.2)

5.2.4.4 Forebay

The standardization charts of forebay and surge tank depend on the parameters presented in Table 5.9. As an example, the standard table for a forebay with Q=0.45 m^3/s is shown in Figure 5.10. The volumetric design values are computed according to standardization curves which are introduced before in the program.

Table 5.9: Required parameters for forebay and surge tank in optimization process (POPEHYE Ver2.2)

Design discharge	Bed type	Headrace pipe length*
Q [m^3/s]	Rock	L' [m]

*: just for surge tank

According to the unit costs of civil works, the cost function based on different discharges will be obtained as shown in Figure 5.10.

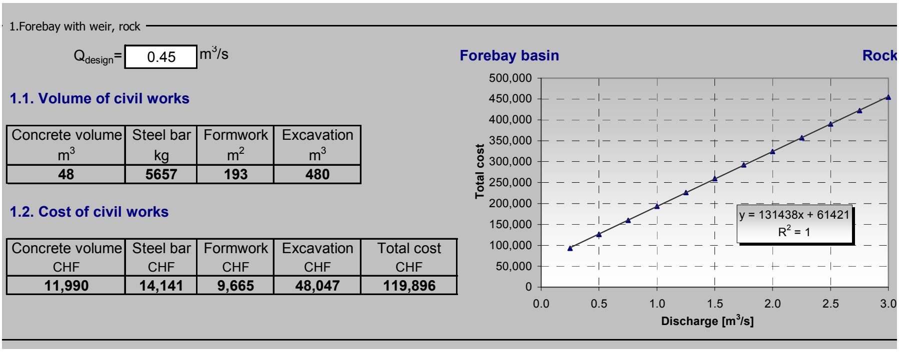


Figure 5.10: Standard format for forebay in optimization process and cost function (POPEHYE Ver2.2)

5.2.4.5 Penstock

The standardization charts of penstock depend on the parameters presented in Table 5.10. As an example, the standard table for a penstock with Q=0.45m^3/s, H=500m, L=600m and energy sale price=0.04CHF/kWh is shown in Figure 5.11. The penstock optimum diameter and wall thickness are computed automatically based on introduced standardized curve in the program and for each discharge.

Table 5.10: Required parameters for penstock in optimization process (POPEHYE Ver2.2)

Design discharge	Energy sale price	Head	Penstock length
Q [m^3/s]	0.04, 0.08, 0.12, 0.16 [CHF/kWh]	H [m]	L [m]

According to the unit cost of steel weight and also installation cost in slope, the cost function based on different discharges will be obtained as illustrated in Figure 5.11.

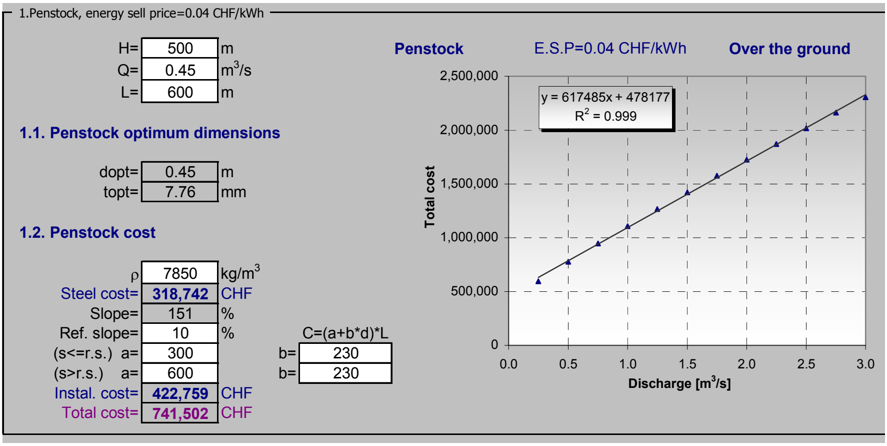


Figure 5.11: Standard format for penstock in optimization process and cost function (POPEHYE Ver2.2)

In this part the volumetric values of civil works related to anchor blocks are also included in the program and cost function of anchors have to be added to penstock component.

5.2.4.6 Power house

The cost of power house depend on different parameters mainly the turbine type. In the program POPEHYE, this cost is estimated with respect to experiences over some constructed SHP projects in Switzerland. So, the cost of power house would be a power function of the installed capacity.

5.2.4.7 Access road

The cost of access road in a power plant project depends on its length and unit cost. In POPEHYE this cost form a very low ratio of total cost of civil works.

5.2.5 Optimization procedure

Total cost of all components of a small power plant can be obtained due to standardization charts and cost functions. These costs are based on volumetric curve according to hydraulic and structural design of structures. After introducing all cost functions, the subsequent steps in optimization process of design discharge are shown in Figure 5.12.

5.2.5.1 Production cost

Annual cost of energy production is obtained from the following simple equation:

$$P_{prod} = \frac{C_{an}}{E} \tag{5.20}$$

C_{an}: annual cost according to annuity factor and amortization time (equation 4.22)

E: annual energy production (kWh/yr) P_{prod}: resale price or production cost (unit cost/yr)

This will show that for the production of every kWh of energy how much funds should be invested for construction of a small power plant.

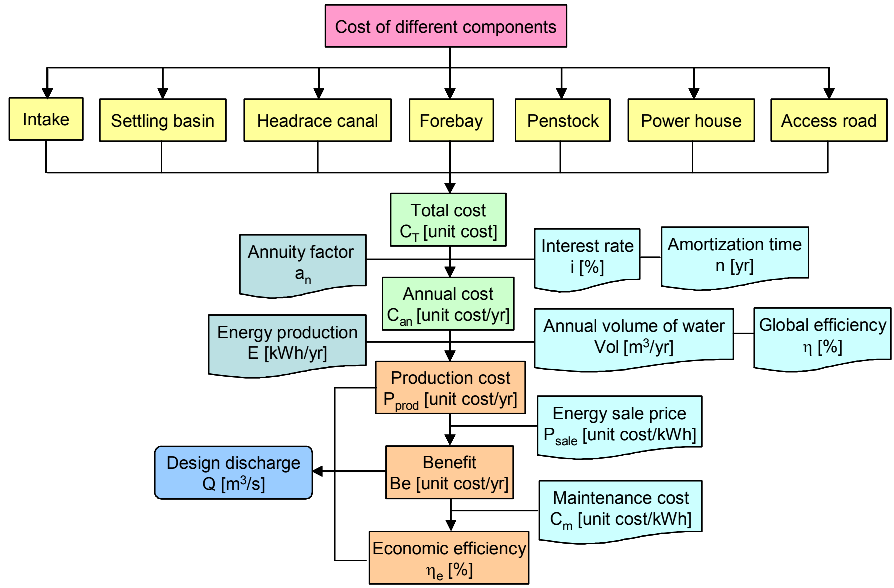


Figure 5.12: Flow charts of optimization process in economical analysis (POPEHYE Ver2.2)

5.2.5.2 Net benefit

Annual net benefit of the SHP is computed as illustrated below:

$$Be = E \cdot P_{sale} - C_{an} \qquad (5.21)$$

P_{sale}: energy sale price (unit cost/kWh) Be: net benefit (unit cost/yr)

5.2.5.3 Economic efficiency

According to the last two steps of annual production cost and benefit, the economic efficiency of the project will be determined as follows:

$$\eta_e = \frac{Be}{C_{an} + C_m} \cdot 100 \quad ; \quad C_m = 900 \cdot P^{0.6} \qquad (5.22)$$

C_m: maintenance cost (unit cost /yr) P: hydraulic power (kW) η_e: economical efficiency (%)

5.2.6 Final results

The final results of economical analysis of the project will be the graphical presentation of the production cost, net benefit and the economic efficiency for all of the discharges according to flow duration curve. This optimization of design discharge (or installed capacity) is performed with a certain and assumed total head.

Figure 5.13 and 5.14 show an example of economical analysis of a small hydro project. According to these Figures, the designer is able to select the optimum design discharge for the project with compromise and judgment through the economic parameters. On the other hand the sensibility analysis should be done with attention to providing minimum production cost (minimum investment that is necessary for each kWh of energy production), maximum net benefit (the benefit which is gained each year by construction of a small plant) and maximum economic efficiency (ratio of net benefit to the cost). However it should be mentioned that having the maximum benefit is the most important factor and it dominates the other two economic features. For the special case presented in the following Figures the optimum choice for the design discharge would be 400 lit/s which corresponds to the maximum annual benefit.

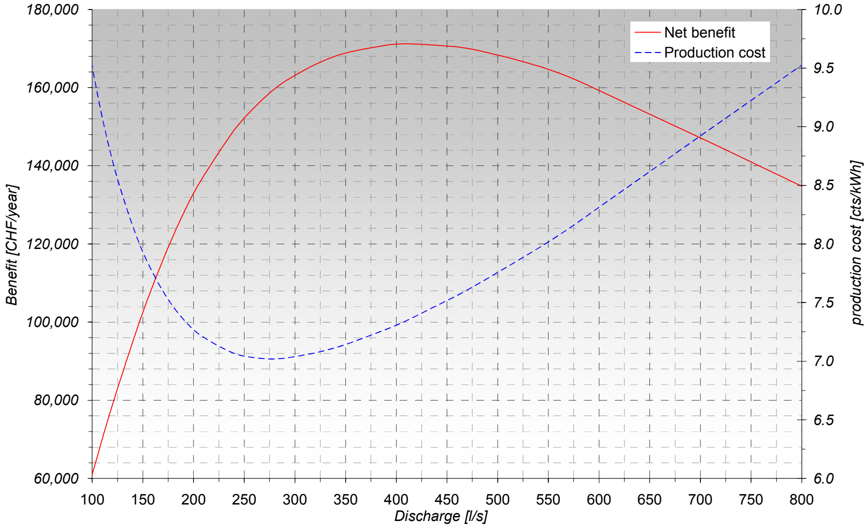


Figure 5.13: An example of final optimization: net benefit and production cost as a function of discharge (POPEHYE Ver2.2)

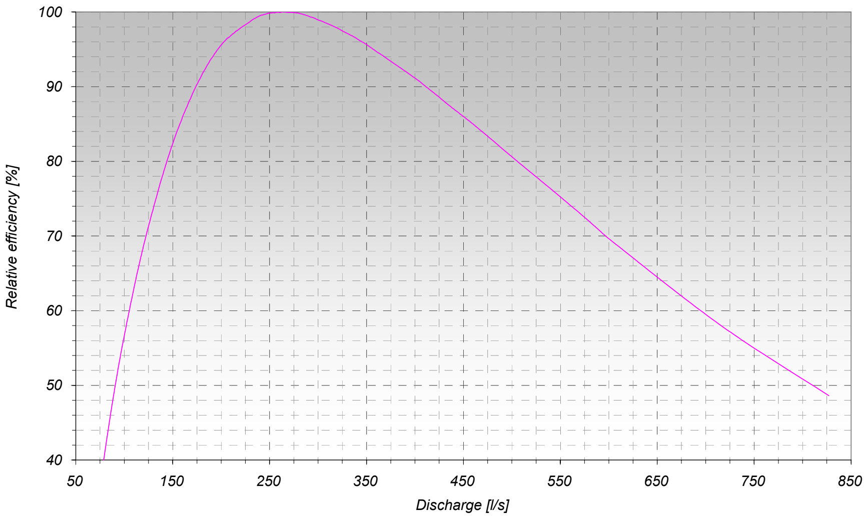


Figure 5.14: An example of final optimization result: economic efficiency as a function of discharge (POPEHYE Ver2.2)

5.3 Pre-dimensioning of the main hydraulic structures

In the previous sections the optimization process of design discharge based on standardization charts and cost functions of different components of SHP was described in detail. The program POPEHYE continues pre dimensioning of the main structures for optimum design discharge. However, detailed design of all components is normally accomplished during standardization procedure and it is just enough to determine the geometry and dimension of structures by using the standard graphs and proposed equations. Nevertheless, in the new version of the program these stages are kept as before because it will help the user to follow up the hydraulic design of different elements of SHP.

6 Environmental impacts

Renewable energy can make a significant contribution to carbon dioxide (CO_2) emissions reduction. It should be accepted that although through having no emissions of CO_2 and other pollutants, electricity production in small hydro plants is environmentally rewarding, the fact is that due to their location in sensitive areas, local impacts are not always negligible.

It is recommended to establish a permanent dialogue with the environmental authorities as a very first step in the design phase. It would be convenient to provide a few guidelines that will help the designer to propose mitigating measures that can be agreed with the authorities.

A high-head SHP project in mountains, being situated in a highly sensitive area, is more likely to generate impact that an integral low head scheme in a valley. The tailwater from the power plant reenters the river and entire areas of the river may be bypassed by a large volume of water, when the plant is in operation.

The exhaustive descriptions of possible impacts due to construction of different components of a small hydropower plant are briefly given in this chapter [8]. With a well arrangement of system of power plant structures, new environmental impacts will not be introduced.

6.1 Construction impacts

The impact generated by construction of hydraulic structures include the loss of ground, the construction and maintenance roads, working platforms, excavation works, blasting and concrete manufacturing plants. Other non negligible impacts are the barrier effect and the alteration of flow consequent to river regulation that did not exist before.

To mitigate such impacts it is recommended that the excavation work should be undertaken in the dry season and the disturbed ground restored as soon as possible. In any case these impacts are always transitory and do not constitute a serious obstacle to the administrative authorization procedure.

In view of its protective role against river erosion, it is wise to restore and reinforce the river bank vegetation that may have been damaged during construction of the structures. The ground should be revegetated with indigenous species, better adapted to the local conditions.

The impact assessment study should take count of the effects of jettisoning excavated material in the stream. Vehicle emissions, excavation dust, the high noise level and other minor burdens contribute to damage the environment, when the scheme is located in sensitive areas. To mitigate the above impacts the traffic operation must be carefully planned.

6.2 Noise impact

The allowable level of noise depends on the local population or on isolated house near to the power house. The noise comes mainly from the turbines and Nowadays noise inside the power house can be reduced, if necessary, by insulation of the power house walls and roof.

6.3 Landscape impact

Each of the components that comprise a hydro scheme has potential to create a change in the visual impact of the site by introducing contrasting forms, lines, color or textures. The design, location, and appearance of any one feature may well determine the level of public acceptance for the entire scheme. Some of these components may be covered by landform or a layer of revegetated terrain.

The penstock is usually the main cause of nuisance. Its layout must be carefully studied using every natural feature such as rocks, ground and vegetation to shroud it and if there is no other solution, painting it so as to minimize contrast with the background. If the penstock can be buried, this is usually a good solution from environmental point of view. Expansion joints and concrete anchor blocks could then be reduced or eliminated; the ground is returned to its original state and the pipe does not form a barrier to the passage of wild life especially in forest.

All component of a SHP should be skilfully inserted into the landscape. Any mitigation strategies should be incorporated in the project, usually without too much extra cost to facilitate permits approval.

6.4 Biological impacts

The reduction on flow in the streambed between the intake and the tailrace downstream of the power house may affect the life of fishes if they exist in the river. In high flow period the water spills over the weir and floods the streambed. Such frequent changes from semi dry to wet condition can make problem for aquatic life.

It must be underlined that if any of the biologic methods for the definition of the residual flow value is implemented, there is a possibility for the developer to decrease the level of the required discharge, by modifying the physical structure of the streambed. Actually growing trees on the riverbanks to provide shadowed areas, deposit gravel in the streambed to improve the substratum, reinforce the riverside shrubs to fight erosion, etc.

A fish passage could be designed to reopen fish migration, which is a firm component of their life cycle. Effective fish passage design for a specific site requires good communication between engineers and biologists and thorough understanding of site characteristics.

7 Conclusion

- Small Hydropower Plants have been recognized as one of the most important energy sources that can provide convenient energy to remote rural communities or industries.
- Civil engineering works of a high-head small hydropower plant contain intake, settling basin, headrace canal or pipe, forebay or surge tank and penstock.
- Standardization of civil works has been established in order to obtain standard design charts and drawings for preliminary stages of a project of small hydropower plant.
- The standardization charts will help designers to evaluate easily and rapidly different alternatives of small hydro in given catchment area, according to head, discharge and location.
- The volumetric standard curves for different components of small hydro allow a first-hand estimate of the total cost of a project by using locally available unit prices of civil works.
- After standardization of all components of small hydro based on civil works, the final volumetric curves and cost functions are implemented in an existing optimization tool called ‘‘POPEHYE”.
- The program “POPEHYE” allows a step by step design and optimization tool for primary evaluation of different alternatives for small hydropower plants.
- The main body of “POPEHYE” is based on economical analysis of different components of project to select the optimum design discharge or hydraulic power for a certain head.
- The present study aims at providing a general guidance with regard to the economical design and the practical realization of the most important components of a small hydropower plant.
- This kind of efforts of standardization and optimization signifies a practical guideline for better realization and implementation of a project.
- With a well system arrangement of power plant structures, new environmental impacts in a small hydropower project will be minimized.

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Mohammadreza Andaroodi

Lausanne, March 2006

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Appendices

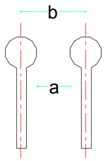
Appendix A

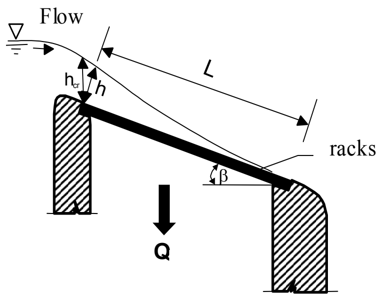
Standardization charts for drop intake (Tyrolean intake)

A1: Design tables of Tyrolean intake

A1.1: Width and length of Tyrolean intake as a function of discharge for a/b=1/3, 1/2, 2/3 and β=30°, 35°, 40°, 45°

a: opening between adjacent bars
b: center spacing of bars
β: slope angle of trashrack (°)
w: river width (m)
Q: design discharge (m^3/s)
B: Trashrack width (m)
L: Trashrack length (m)





Values = c * Q ^{d}

Values: width [B] , length [L]

a/b=1/3 , β=30°

Values	c	d
B	1.9972	0.3994
L	3.3304	0.4004

a/b=1/3 , β=35°

Values	c	d
B	2.1378	0.3987
L	3.5574	0.4009

a/b=1/2 , β=30°

Values	c	d
B	1.5644	0.3994
L	2.6129	0.4004

a/b=1/2 , β=35°

Values	c	d
B	1.6730	0.4004
L	2.7926	0.3998

a/b=2/3 , β=30°

Values	c	d
B	1.3188	0.3993
L	2.1959	0.4004

a/b=2/3 , β=35°

Values	c	d
B	1.4100	0.3985
L	2.3474	0.4010

a/b=1/3 , β=40°

Values	c	d
B	2.3159	0.4000
L	3.8564	0.4000

a/b=1/3 , β=45°

Values	c	d
B	2.5472	0.4007
L	4.2478	0.3995

a/b=1/2 , β=40°

Values	c	d
B	1.8141	0.4007
L	3.0255	0.3995

a/b=1/2 , β=45°

Values	c	d
B	1.9972	0.3994
L	3.3304	0.4004

a/b=2/3 , β=40°

Values	c	d
B	1.5281	0.3987
L	2.5441	0.4008

a/b=2/3 , β=45°

Values	c	d
B	1.6818	0.3990
L	2.8011	0.4007

A1.2: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge for a/b=1/3 and β=30°

$\beta=30^o$
a/b=1/3

Values = c * W + d

Values: concrete volume, reinforcment, excavation and formwork

River bed: rock

Concrete volume [m³]

Q	c	d
0.25	1.74	22.7
0.50	2.22	33.7
0.75	2.69	45.2
1.00	3.13	54.2
1.50	3.70	68.6
2.00	4.61	89.4
2.50	5.62	118.3
3.00	6.64	144.2

Reinforcement [kg]

Q	c	d
0.25	255.06	3422.3
0.50	318.51	6070.6
0.75	361.60	7778.0
1.00	458.55	10072.0
1.50	566.83	13440.0
2.00	709.83	17460.0
2.50	853.54	22375.0
3.00	992.78	26647.0

Excavation [m³]

Q	c	d
0.25	3.18	26.1
0.50	3.73	35.8
0.75	4.18	44.7
1.00	4.44	51.2
1.50	4.86	61.8
2.00	5.32	73.9
2.50	5.86	88.5
3.00	6.24	100.5

Formwork [m²]

Q	c	d
0.25	6.97	55.0
0.50	8.07	76.9
0.75	8.96	96.9
1.00	9.63	110.8
1.50	10.58	134.2
2.00	11.52	159.3
2.50	12.49	190.8
3.00	13.28	215.1

River bed: alluvium

Concrete volume [m³]

Q	c	d
0.25	3.33	39.0
0.50	4.09	55.3
0.75	4.78	71.7
1.00	5.35	84.3
1.50	6.13	104.4
2.00	7.27	131.7
2.50	8.55	168.4
3.00	9.76	200.7

Reinforcement [kg]

Q	c	d
0.25	487.75	5799.1
0.50	586.20	9173.7
0.75	642.41	11345.0
1.00	783.71	14475.0
1.50	938.85	18917.0
2.00	1119.30	23970.0
2.50	1298.30	29984.0
3.00	1459.20	35091.0

Excavation [m³]

Q	c	d
0.25	5.96	46.0
0.50	7.00	63.6
0.75	7.83	79.9
1.00	8.33	91.9
1.50	9.11	111.4
2.00	9.97	133.6
2.50	10.98	160.4
3.00	11.70	182.5

Formwork [m²]

Q	c	d
0.25	10.46	82.5
0.50	12.11	115.3
0.75	13.44	145.4
1.00	14.45	166.2
1.50	15.87	201.3
2.00	17.28	239.0
2.50	18.73	286.2
3.00	19.92	322.7

A1.3: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge for a/b=1/3 and $\beta=35^o$

$\beta=35^o$
a/b=1/3

Values = c * W + d

Values: concrete volume, reinforcment, excavation and formwork

River bed: rock

Concrete volume [m^3]

Q	c	d
0.25	1.87	25.7
0.50	2.41	38.7
0.75	2.96	55.3
1.00	3.50	70.4
1.50	4.16	89.5
2.00	5.18	116.2
2.50	6.31	152.1
3.00	7.46	184.7

Reinforcement [kg]

Q	c	d
0.25	274.01	3879.3
0.50	345.26	6957.7
0.75	398.54	9378.2
1.00	512.86	12743.0
1.50	636.87	17049.0
2.00	798.42	22101.0
2.50	959.07	28132.0
3.00	1115.00	33421.0

Excavation [m^3]

Q	c	d
0.25	3.28	29.0
0.50	3.86	40.3
0.75	4.45	52.7
1.00	4.85	62.9
1.50	5.29	76.0
2.00	5.77	90.6
2.50	6.34	108.0
3.00	6.74	122.4

Formwork [m^2]

Q	c	d
0.25	7.49	61.3
0.50	8.75	86.9
0.75	9.88	115.8
1.00	10.77	138.5
1.50	11.89	167.8
2.00	12.96	198.7
2.50	14.03	236.5
3.00	14.92	266.2

River bed: alluvium

Concrete volume [m^3]

Q	c	d
0.25	3.51	43.5
0.50	4.34	62.7
0.75	5.19	86.1
1.00	5.93	106.7
1.50	6.81	132.8
2.00	8.07	167.2
2.50	9.48	212.5
3.00	10.83	252.6

Reinforcement [kg]

Q	c	d
0.25	513.72	6483.0
0.50	622.42	10405.0
0.75	697.91	13524.0
1.00	867.90	18057.0
1.50	1042.00	23675.0
2.00	1243.00	29968.0
2.50	1440.30	37294.0
3.00	1619.10	43576.0

Excavation [m^3]

Q	c	d
0.25	6.14	51.4
0.50	7.25	72.0
0.75	8.35	94.7
1.00	9.09	113.3
1.50	9.92	137.5
2.00	10.82	164.5
2.50	11.88	196.5
3.00	12.65	223.1

Formwork [m^2]

Q	c	d
0.25	11.23	92.0
0.50	13.13	130.4
0.75	14.82	173.6
1.00	16.16	207.8
1.50	17.83	251.7
2.00	19.44	298.0
2.50	21.05	354.8
3.00	22.37	399.2

A1.4: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge for a/b=1/3 and β=40°

β=40°
a/b=1/3

Values = c * W + d

Values: concrete volume, reinforcment, excavation and formwork

River bed: rock

Concrete volume [m^3]

Q	c	d
0.25	2.02	29.5
0.50	2.63	45.0
0.75	3.28	68.1
1.00	3.88	86.6
1.50	4.68	116.3
2.00	5.83	150.7
2.50	7.10	195.7
3.00	8.46	246.1

Reinforcement [kg]

Q	c	d
0.25	296.25	4463.4
0.50	377.05	8107.7
0.75	441.10	11435.0
1.00	568.48	15544.0
1.50	716.03	21737.0
2.00	897.30	28150.0
2.50	1078.90	35613.0
3.00	1264.70	43597.0

Excavation [m^3]

Q	c	d
0.25	3.40	32.7
0.50	4.02	46.0
0.75	4.76	62.7
1.00	5.18	74.8
1.50	5.78	93.7
2.00	6.31	111.8
2.50	6.89	132.5
3.00	7.43	153.4

Formwork [m^2]

Q	c	d
0.25	8.10	69.4
0.50	9.56	99.6
0.75	10.93	139.2
1.00	11.94	166.6
1.50	13.36	210.3
2.00	14.56	249.1
2.50	15.79	294.5
3.00	16.92	340.1

River bed: alluvium

Concrete volume [m^3]

Q	c	d
0.25	3.72	49.3
0.50	4.64	72.0
0.75	5.66	104.2
1.00	6.47	129.2
1.50	7.57	169.0
2.00	8.98	212.9
2.50	10.55	268.8
3.00	12.17	330.2

Reinforcement [kg]

Q	c	d
0.25	544.74	7351.3
0.50	665.40	11985.0
0.75	761.45	16294.0
1.00	948.13	21781.0
1.50	1158.80	29796.0
2.00	1383.40	37729.0
2.50	1602.00	46720.0
3.00	1820.00	56173.0

Excavation [m^3]

Q	c	d
0.25	6.37	58.1
0.50	7.54	82.5
0.75	8.93	113.1
1.00	9.72	135.4
1.50	10.85	170.3
2.00	11.84	203.6
2.50	12.92	241.9
3.00	13.93	280.6

Formwork [m^2]

Q	c	d
0.25	12.15	104.0
0.50	14.34	149.3
0.75	16.40	208.8
1.00	17.91	249.9
1.50	20.05	315.5
2.00	21.85	373.6
2.50	23.68	441.8
3.00	25.38	510.1

A1.5: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge for a/b=1/3 and β=45°

$\beta=45^{o}$
a/b=1/3

Values = c * W + d

Values: concrete volume, reinforcment, excavation and formwork

River bed: rock

Concrete volume [m^3]

Q	c	d
0.25	2.21	34.5
0.50	2.92	56.5
0.75	3.62	80.8
1.00	4.29	103.0
1.50	5.23	145.4
2.00	6.59	196.3
2.50	8.02	253.4
3.00	9.56	316.7

Reinforcement [kg]

Q	c	d
0.25	322.91	5052.1
0.50	419.13	8105.3
0.75	486.81	10862.0
1.00	628.52	15090.0
1.50	801.05	22265.0
2.00	1015.00	30239.0
2.50	1218.70	38482.0
3.00	1428.70	47345.0

Excavation [m^3]

Q	c	d
0.25	3.55	37.4
0.50	4.36	55.7
0.75	5.00	73.0
1.00	5.46	87.4
1.50	6.24	113.5
2.00	6.91	138.9
2.50	7.54	164.2
3.00	8.10	189.5

Formwork [m^2]

Q	c	d
0.25	8.83	79.9
0.50	10.62	122.3
0.75	12.07	162.7
1.00	13.20	195.5
1.50	14.95	257.3
2.00	16.47	314.0
2.50	17.83	370.3
3.00	19.11	415.7

River bed: alluvium

Concrete volume [m^3]

Q	c	d
0.25	3.98	56.8
0.50	5.10	88.7
0.75	6.12	122.3
1.00	7.02	152.2
1.50	8.35	208.4
2.00	10.05	272.7
2.50	11.79	343.0
3.00	13.61	419.5

Reinforcement [kg]

Q	c	d
0.25	582.82	8309.4
0.50	731.58	12723.0
0.75	823.31	16442.0
1.00	1028.40	22289.0
1.50	1278.70	31907.0
2.00	1547.40	42000.0
2.50	1791.10	52096.0
3.00	2034.20	62718.0

Excavation [m^3]

Q	c	d
0.25	6.66	66.8
0.50	8.17	100.3
0.75	9.38	132.1
1.00	10.24	158.7
1.50	11.70	206.9
2.00	12.96	253.9
2.50	14.13	300.8
3.00	15.19	347.7

Formwork [m^2]

Q	c	d
0.25	13.24	119.9
0.50	15.94	183.5
0.75	18.10	244.0
1.00	19.81	293.3
1.50	22.43	385.9
2.00	24.71	471.1
2.50	26.75	555.5
3.00	28.67	623.5

A1.6: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge for a/b=1/2 and β=30°

β=30°
a/b=1/2

Values = c * W + d

Values: concrete volume, reinforcment, excavation and formwork

River bed: rock

Concrete volume [m^3]

Q	c	d
0.25	1.62	18.8
0.50	2.00	25.3
0.75	2.42	33.5
1.00	2.81	39.7
1.50	3.35	52.8
2.00	4.11	64.0
2.50	4.95	80.0
3.00	5.91	102.9

Reinforcement [kg]

Q	c	d
0.25	237.04	2852.6
0.50	287.61	4124.6
0.75	326.02	5190.9
1.00	412.30	6680.8
1.50	512.64	9261.9
2.00	632.35	11345.0
2.50	751.83	13921.0
3.00	883.77	17423.0

Excavation [m^3]

Q	c	d
0.25	2.88	21.9
0.50	3.23	27.8
0.75	3.59	34.4
1.00	3.79	39.2
1.50	4.25	48.7
2.00	4.51	55.6
2.50	4.87	64.6
3.00	5.32	75.9

Formwork [m^2]

Q	c	d
0.25	6.48	45.0
0.50	7.29	57.4
0.75	8.08	71.9
1.00	8.66	81.3
1.50	9.57	102.6
2.00	10.26	115.1
2.50	11.00	132.5
3.00	11.82	155.8

River bed: alluvium

Concrete volume [m^3]

Q	c	d
0.25	3.06	32.7
0.50	3.62	42.4
0.75	4.22	54.3
1.00	4.71	63.1
1.50	5.47	81.3
2.00	6.36	96.3
2.50	7.39	117.1
3.00	8.57	146.1

Reinforcement [kg]

Q	c	d
0.25	447.78	4875.4
0.50	519.15	6581.9
0.75	567.30	7989.8
1.00	690.01	10104.0
1.50	837.81	13638.0
2.00	979.86	16324.0
2.50	1121.80	19563.0
3.00	1281.10	23892.0

Excavation [m^3]

Q	c	d
0.25	5.40	38.3
0.50	6.05	49.1
0.75	6.73	61.2
1.00	7.11	69.9
1.50	7.97	87.3
2.00	8.46	100.1
2.50	9.14	116.5
3.00	9.97	137.3

Formwork [m^2]

Q	c	d
0.25	9.72	67.5
0.50	10.94	86.2
0.75	12.12	107.9
1.00	12.99	122.0
1.50	14.35	153.9
2.00	15.39	172.6
2.50	16.50	198.7
3.00	17.73	233.7

A1.7: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge for a/b=1/2 and β=35°

$\beta=35^o$
a/b=1/2

Values = c * W + d

Values: concrete volume, reinforcment, excavation and formwork

River bed: rock

Concrete volume [m³]

Q	c	d
0.25	1.72	21.0
0.50	2.15	28.6
0.75	2.65	40.6
1.00	3.08	48.4
1.50	3.68	64.3
2.00	4.52	78.5
2.50	5.47	97.8
3.00	6.52	125.0

Reinforcement [kg]

Q	c	d
0.25	252.06	3165.3
0.50	308.91	4670.7
0.75	356.19	6247.3
1.00	451.70	8070.7
1.50	563.56	11203.0
2.00	696.79	13782.0
2.50	830.60	16878.0
3.00	974.77	21027.0

Excavation [m³]

Q	c	d
0.25	2.95	23.9
0.50	3.32	30.9
0.75	3.83	40.3
1.00	4.04	45.9
1.50	4.52	57.2
2.00	4.81	65.6
2.50	5.17	76.0
3.00	5.64	89.1

Formwork [m²]

Q	c	d
0.25	6.89	49.4
0.50	7.83	64.0
0.75	8.83	85.5
1.00	9.49	96.9
1.50	10.52	122.3
2.00	11.31	137.8
2.50	12.15	158.1
3.00	13.04	185.5

River bed: alluvium

Concrete volume [m³]

Q	c	d
0.25	3.20	35.9
0.50	3.82	47.3
0.75	4.56	64.6
1.00	5.11	75.4
1.50	5.94	97.5
2.00	6.93	116.1
2.50	8.05	141.0
3.00	9.34	175.2

Reinforcement [kg]

Q	c	d
0.25	468.06	5349.5
0.50	547.34	7362.0
0.75	613.60	9474.1
1.00	747.86	12028.0
1.50	909.85	16275.0
2.00	1067.40	19577.0
2.50	1223.40	23434.0
3.00	1396.40	28531.0

Excavation [m³]

Q	c	d
0.25	5.54	42.1
0.50	6.23	54.8
0.75	7.18	72.0
1.00	7.58	82.4
1.50	8.48	103.0
2.00	9.02	118.5
2.50	9.70	137.6
3.00	10.58	161.8

Formwork [m²]

Q	c	d
0.25	10.33	74.1
0.50	11.75	96.1
0.75	13.24	128.3
1.00	14.23	145.4
1.50	15.78	183.4
2.00	16.96	206.7
2.50	18.23	237.2
3.00	19.56	278.2

A1.8: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge for a/b=1/2 and $\beta=40^o$

$\beta=40^o$
a/b=1/2

Values = c * W + d

Values: concrete volume, reinforcment, excavation and formwork

River bed: rock

Concrete volume [m^3]

Q	c	d
0.25	1.84	23.7
0.50	2.36	35.0
0.75	2.90	49.6
1.00	3.39	59.3
1.50	4.06	79.0
2.00	5.01	96.8
2.50	6.05	120.7
3.00	7.23	153.3

Reinforcement [kg]

Q	c	d
0.25	269.42	3572.5
0.50	338.71	5690.0
0.75	390.35	7585.4
1.00	95.89	9839.6
1.50	621.46	13684.0
2.00	771.73	16903.0
2.50	919.25	20698.0
3.00	1080.30	25650.0

Excavation [m^3]

Q	c	d
0.25	3.05	26.5
0.50	3.56	36.5
0.75	4.10	47.6
1.00	4.34	54.4
1.50	4.85	67.9
2.00	5.15	78.0
2.50	5.54	90.4
3.00	6.01	105.6

Formwork [m^2]

Q	c	d
0.25	7.36	55.0
0.50	8.59	77.0
0.75	9.67	102.5
1.00	10.42	116.7
1.50	11.60	147.1
2.00	12.53	166.3
2.50	13.45	191.2
3.00	14.45	222.9

River bed: alluvium

Concrete volume [m^3]

Q	c	d
0.25	3.37	40.0
0.50	4.14	56.8
0.75	4.95	77.5
1.00	5.56	90.9
1.50	6.48	117.8
2.00	7.58	141.0
2.50	8.82	171.4
3.00	10.23	212.1

Reinforcement [kg]

Q	c	d
0.25	492.46	5960.1
0.50	594.35	8817.0
0.75	666.32	11337.0
1.00	814.03	14459.0
1.50	992.56	19620.0
2.00	1168.20	23705.0
2.50	1340.30	28407.0
3.00	1529.70	34443.0

Excavation [m^3]

Q	c	d
0.25	5.72	46.9
0.50	6.68	65.0
0.75	7.70	85.4
1.00	8.15	97.9
1.50	9.09	122.7
2.00	9.65	141.5
2.50	10.40	164.4
3.00	11.27	192.4

Formwork [m^2]

Q	c	d
0.25	11.05	82.5
0.50	12.88	115.5
0.75	14.51	153.8
1.00	15.63	175.0
1.50	17.40	220.6
2.00	18.79	249.4
2.50	20.17	286.8
3.00	21.68	334.3

A1.9: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge for a/b=1/2 and β=45°

β=45°
a/b=1/2

Values = c * W + d

Values: concrete volume, reinforcment, excavation and formwork

River bed: rock

Concrete volume [m^3]

Q	c	d
0.25	1.98	27.1
0.50	2.57	40.8
0.75	3.17	57.9
1.00	3.75	73.5
1.50	4.51	98.2
2.00	5.63	126.7
2.50	6.80	157.5
3.00	8.10	198.6

Reinforcement [kg]

Q	c	d
0.25	289.79	4102.6
0.50	368.08	6654.1
0.75	425.97	8899.6
1.00	548.70	12155.0
1.50	690.41	16950.0
2.00	866.78	21925.0
2.50	1033.20	26789.0
3.00	1211.70	33000.0

Excavation [m^3]

Q	c	d
0.25	3.18	29.9
0.50	3.73	41.7
0.75	4.30	54.5
1.00	4.68	65.1
1.50	5.22	81.5
2.00	5.68	97.0
2.50	6.10	112.1
3.00	6.60	130.4

Formwork [m^2]

Q	c	d
0.25	7.92	62.4
0.50	9.33	88.6
0.75	10.56	118.2
1.00	11.53	141.7
1.50	12.89	178.9
2.00	14.07	211.2
2.50	15.12	242.3
3.00	16.21	281.2

River bed: alluvium

Concrete volume [m^3]

Q	c	d
0.25	3.57	45.2
0.50	4.43	65.3
0.75	5.32	89.5
1.00	6.09	110.7
1.50	7.12	144.2
2.00	8.47	180.9
2.50	9.85	219.7
3.00	11.41	270.4

Reinforcement [kg]

Q	c	d
0.25	522.48	6755.4
0.50	635.77	10178.0
0.75	714.85	13145.0
1.00	891.44	17608.0
1.50	1090.00	23985.0
2.00	1303.90	30268.0
2.50	1496.20	36231.0
3.00	1705.10	43737.0

Excavation [m^3]

Q	c	d
0.25	5.96	53.1
0.50	7.00	74.6
0.75	8.06	98.2
1.00	8.78	117.7
1.50	9.79	147.9
2.00	10.64	176.5
2.50	11.43	204.6
3.00	12.38	238.4

Formwork [m^2]

Q	c	d
0.25	11.88	93.6
0.50	14.00	132.9
0.75	15.84	177.3
1.00	17.29	212.5
1.50	19.33	268.4
2.00	21.10	268.4
2.50	22.68	363.4
3.00	24.31	421.7

A1.10: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge for a/b=2/3 and β=30°

β=30°
a/b=2/3

Values = c * W + d

Values: concrete volume, reinforcment, excavation and formwork

River bed: rock

Concrete volume [m^3]

Q	c	d
0.25	1.55	16.8
0.50	1.90	22.0
0.75	2.29	28.9
1.00	2.65	34.0
1.50	3.14	44.6
2.00	3.84	53.6
2.50	4.63	66.5
3.00	5.52	85.4

Reinforcement [kg]

Q	c	d
0.25	226.47	2550.4
0.50	272.86	3327.6
0.75	308.09	4116.4
1.00	388.13	5269.6
1.50	481.20	7231.4
2.00	591.34	8758.2
2.50	703.01	10701.0
3.00	825.62	13455.0

Excavation [m^3]

Q	c	d
0.25	2.71	19.6
0.50	3.00	24.4
0.75	3.32	30.1
1.00	3.50	34.0
1.50	3.90	41.8
2.00	4.13	47.5
2.50	4.44	54.9
3.00	4.86	64.7

Formwork [m^2]

Q	c	d
0.25	6.19	39.7
0.50	6.92	49.5
0.75	7.64	61.5
1.00	8.15	69.0
1.50	8.98	86.2
2.00	9.60	95.7
2.50	10.29	109.5
3.00	11.05	129.2

River bed: alluvium

Concrete volume [m^3]

Q	c	d
0.25	2.90	29.3
0.50	3.40	37.2
0.75	3.95	47.2
1.00	4.40	54.5
1.50	5.09	69.4
2.00	5.90	81.4
2.50	6.85	98.4
3.00	7.95	122.5

Reinforcement [kg]

Q	c	d
0.25	424.92	4383.6
0.50	488.04	5510.8
0.75	531.61	6585.2
1.00	644.75	8269.9
1.50	779.73	11030.0
2.00	909.27	13049.0
2.50	1040.20	15542.0
3.00	1188.90	19016.0

Excavation [m^3]

Q	c	d
0.25	5.09	34.3
0.50	5.63	43.0
0.75	6.23	53.3
1.00	6.57	60.4
1.50	7.31	74.8
2.00	7.74	85.1
2.50	8.33	98.7
3.00	9.11	116.7

Formwork [m^2]

Q	c	d
0.25	9.29	59.5
0.50	10.38	74.2
0.75	11.45	92.2
1.00	12.23	103.5
1.50	13.47	129.4
2.00	14.40	143.6
2.50	15.43	164.3
3.00	16.57	193.8

A1.11: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge for a/b=2/3 and β=35°

β=35°
a/b=2/3

Values = c * W + d

Values: concrete volume, reinforcment, excavation and formwork

River bed: rock

Concrete volume [m³]

Q	c	d
0.25	1.63	18.5
0.50	2.03	24.6
0.75	2.45	32.5
1.00	2.84	38.4
1.50	3.39	50.7
2.00	4.15	61.3
2.50	5.07	80.6
3.00	6.05	102.8

Reinforcement [kg]

Q	c	d
0.25	239.12	2797.6
0.50	290.63	3724.5
0.75	329.04	4634.0
1.00	416.60	5960.0
1.50	518.56	8225.7
2.00	639.85	10017.0
2.50	770.22	12934.0
3.00	905.03	16156.0

Excavation [m³]

Q	c	d
0.25	2.77	21.3
0.50	3.08	26.9
0.75	3.43	33.3
1.00	3.61	37.7
1.50	4.03	46.8
2.00	4.27	53.3
2.50	4.72	64.1
3.00	5.14	75.2

Formwork [m²]

Q	c	d
0.25	6.54	43.1
0.50	7.37	54.6
0.75	8.16	68.4
1.00	8.75	77.0
1.50	9.68	96.9
2.00	10.39	108.2
2.50	11.27	130.1
3.00	12.11	152.5

River bed: alluvium

Concrete volume [m³]

Q	c	d
0.25	3.02	31.9
0.50	3.57	41.1
0.75	4.16	52.5
1.00	4.65	60.9
1.50	5.40	78.1
2.00	6.29	92.2
2.50	7.43	117.4
3.00	8.62	145.5

Reinforcement [kg]

Q	c	d
0.25	441.95	4759.0
0.50	511.84	6093.3
0.75	559.82	7332.8
1.00	681.12	9250.5
1.50	827.20	12422.0
2.00	968.88	14782.0
2.50	1128.40	18516.0
3.00	1289.00	22542.0

Excavation [m³]

Q	c	d
0.25	5.20	37.3
0.50	5.78	47.5
0.75	6.44	59.2
1.00	6.77	67.3
1.50	7.56	83.9
2.00	8.01	96.0
2.50	8.84	115.7
3.00	9.63	136.1

Formwork [m²]

Q	c	d
0.25	9.80	64.7
0.50	11.05	82.0
0.75	12.23	102.7
1.00	13.13	115.5
1.50	14.52	145.4
2.00	15.58	162.3
2.50	16.90	195.1
3.00	18.16	228.7

A1.12: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge for a/b=2/3 and β=40°

β=40°
a/b=2/3

Values = c * W + d

Values: concrete volume, reinforcment, excavation and formwork

River bed: rock

Concrete volume [m^3]

Q	c	d
0.25	1.73	20.6
0.50	2.17	27.9
0.75	2.67	39.4
1.00	3.11	46.8
1.50	3.72	62.0
2.00	4.57	75.3
2.50	5.57	98.6
3.00	6.65	125.0

Reinforcement [kg]

Q	c	d
0.25	253.55	3106.4
0.50	311.31	4227.2
0.75	359.15	5614.0
1.00	455.15	7252.0
1.50	569.48	10026.0
2.00	704.23	12274.0
2.50	845.71	15778.0
3.00	993.67	19615.0

Excavation [m^3]

Q	c	d
0.25	2.86	23.3
0.50	3.19	29.9
0.75	3.67	39.0
1.00	3.88	44.4
1.50	4.31	55.1
2.00	4.57	63.0
2.50	5.05	75.6
3.00	5.48	88.4

Formwork [m^2]

Q	c	d
0.25	6.93	47.5
0.50	7.89	61.2
0.75	8.90	81.7
1.00	9.56	92.4
1.50	10.63	116.0
2.00	11.43	130.3
2.50	12.37	156.4
3.00	13.29	182.4

River bed: alluvium

Concrete volume [m^3]

Q	c	d
0.25	3.16	35.1
0.50	3.77	46.0
0.75	4.51	62.6
1.00	5.05	72.9
1.50	5.87	93.8
2.00	6.86	111.4
2.50	8.09	141.4
3.00	9.39	174.7

Reinforcement [kg]

Q	c	d
0.25	462.53	5230.2
0.50	540.27	6829.1
0.75	606.07	8731.8
1.00	739.01	11070.0
1.50	899.24	14902.0
2.00	1056.40	17833.0
2.50	1229.40	22288.0
3.00	1403.60	27041.0

Excavation [m^3]

Q	c	d
0.25	5.36	41.0
0.50	5.99	53.1
0.75	6.89	69.7
1.00	7.27	79.6
1.50	8.08	99.2
2.00	8.57	113.9
2.50	9.47	137.0
3.00	10.28	160.6

Formwork [m^2]

Q	c	d
0.25	10.40	71.3
0.50	11.84	91.8
0.75	13.35	122.6
1.00	14.34	138.7
1.50	15.94	174.0
2.00	17.14	195.4
2.50	18.56	234.6
3.00	19.94	273.6

A1.13: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge for a/b=2/3 and β=45°

β=45°
a/b=2/3

Values = c * W + d

Values: concrete volume, reinforcment, excavation and formwork

River bed: rock

Concrete volume [m³]

Q	c	d
0.25	1.85	23.3
0.50	2.35	32.2
0.75	2.89	45.6
1.00	3.42	57.7
1.50	4.14	80.6
2.00	5.10	98.4
2.50	6.17	121.9
3.00	7.35	153.9

Reinforcement [kg]

Q	c	d
0.25	270.83	3519.3
0.50	336.56	4886.9
0.75	388.74	6511.0
1.00	500.64	8923.8
1.50	633.70	12988.0
2.00	785.63	15972.0
2.50	937.09	19489.0
3.00	1098.60	24138.0

Excavation [m³]

Q	c	d
0.25	2.96	25.9
0.50	3.32	33.8
0.75	3.84	44.3
1.00	4.18	52.8
1.50	4.78	68.2
2.00	5.06	78.2
2.50	5.42	90.2
3.00	5.89	105.3

Formwork [m²]

Q	c	d
0.25	7.40	53.3
0.50	8.53	69.7
0.75	9.64	93.6
1.00	10.52	112.0
1.50	11.83	147.4
2.00	12.75	166.0
2.50	13.71	189.8
3.00	14.70	221.1

River bed: alluvium

Concrete volume [m³]

Q	c	d
0.25	3.33	39.2
0.50	4.01	52.4
0.75	4.81	71.6
1.00	5.51	88.3
1.50	6.53	119.5
2.00	7.63	142.6
2.50	8.88	172.5
3.00	10.29	212.5

Reinforcement [kg]

Q	c	d
0.25	487.72	5851.5
0.50	574.99	7787.1
0.75	646.96	10008.0
1.00	806.47	13403.0
1.50	999.29	18941.0
2.00	1175.70	22774.0
2.50	1349.00	27166.0
3.00	1539.00	32890.0

Excavation [m³]

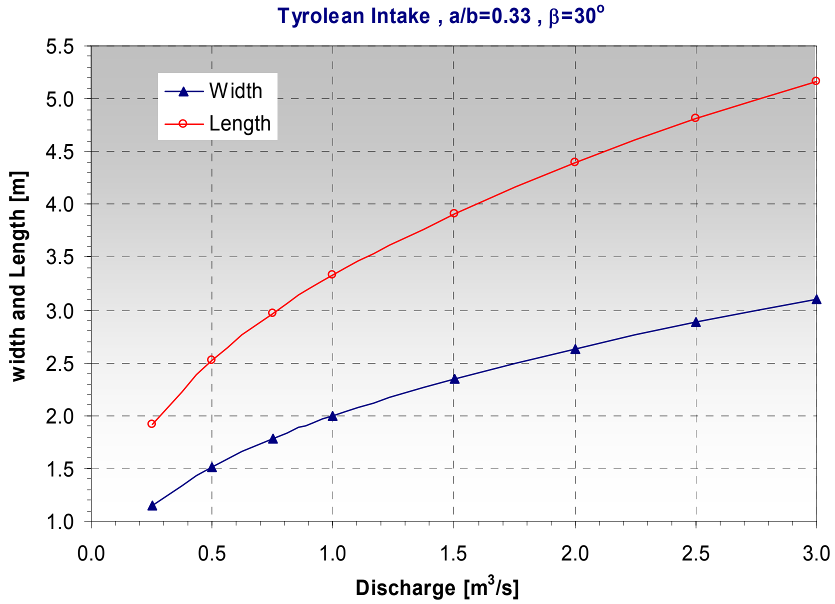
Q	c	d
0.25	5.56	45.9
0.50	6.23	60.2
0.75	7.20	79.5
1.00	7.83	95.1
1.50	8.96	123.4
2.00	9.50	141.8
2.50	10.17	164.1
3.00	11.05	191.9

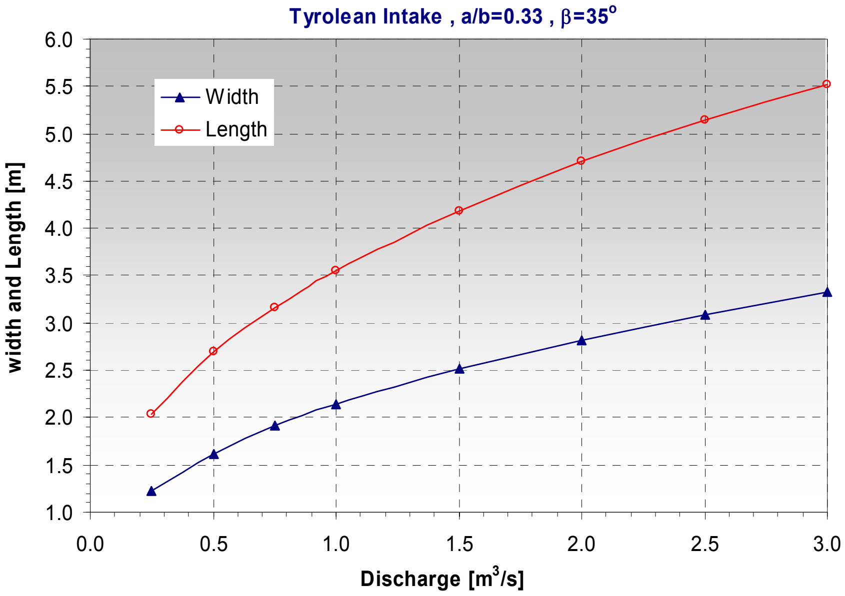
Formwork [m²]

Q	c	d
0.25	11.10	80.0
0.50	12.80	104.6
0.75	14.45	140.4
1.00	15.78	168.0
1.50	17.74	221.2
2.00	19.13	249.1
2.50	20.57	284.6
3.00	22.05	331.6

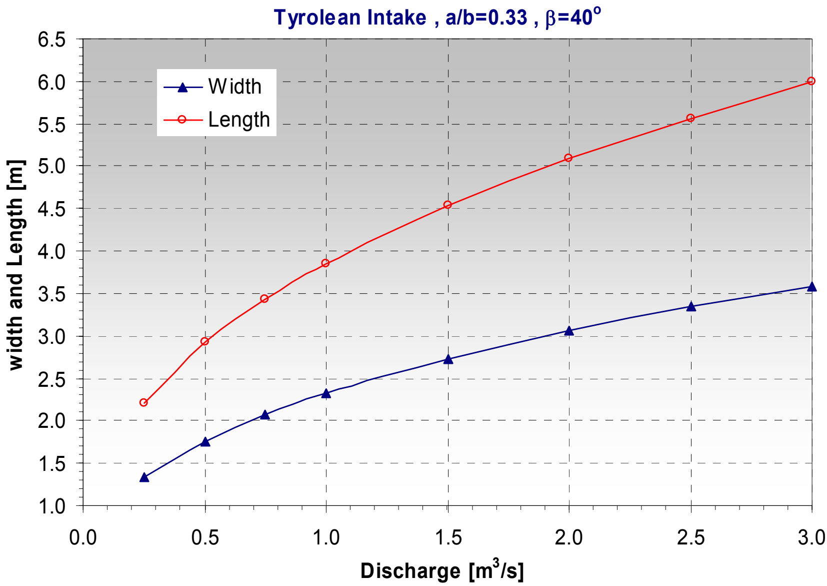
A2: Design charts of Tyrolean intake

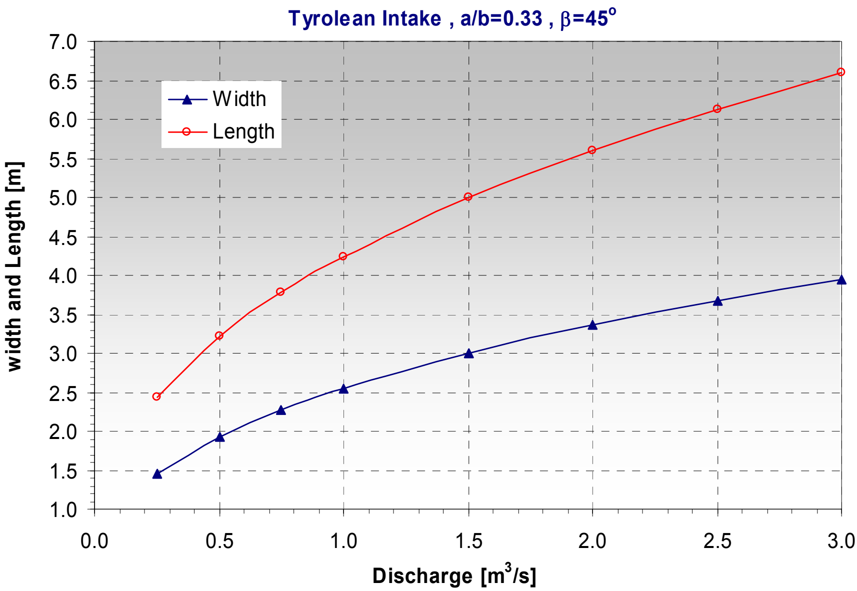
A2.1: Width and length of Tyrolean intake as a function of discharge, a/b and β



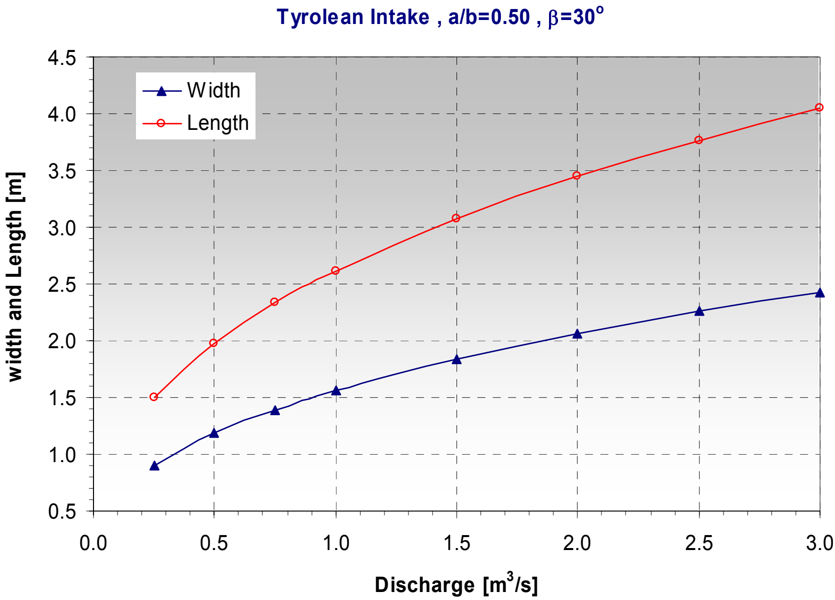


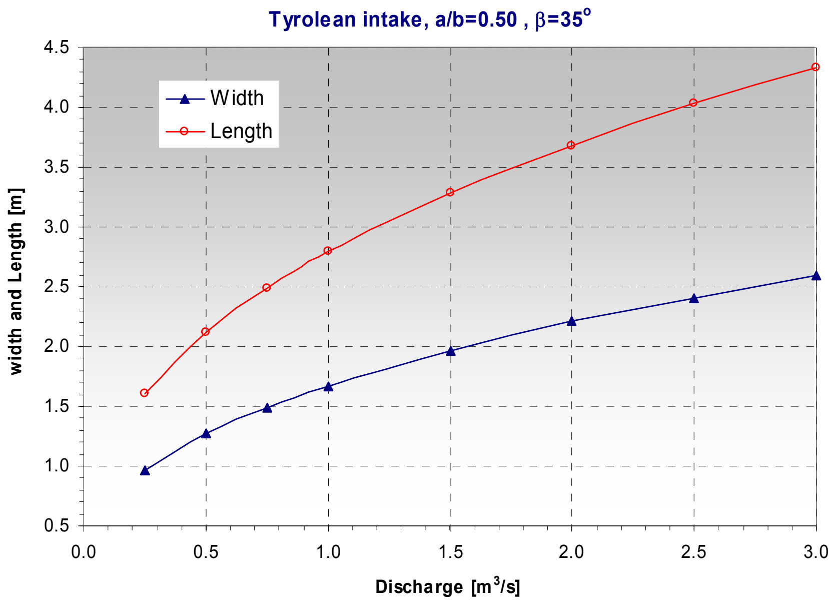
A2.1: Width and length of Tyrolean intake as a function of discharge, a/b and β



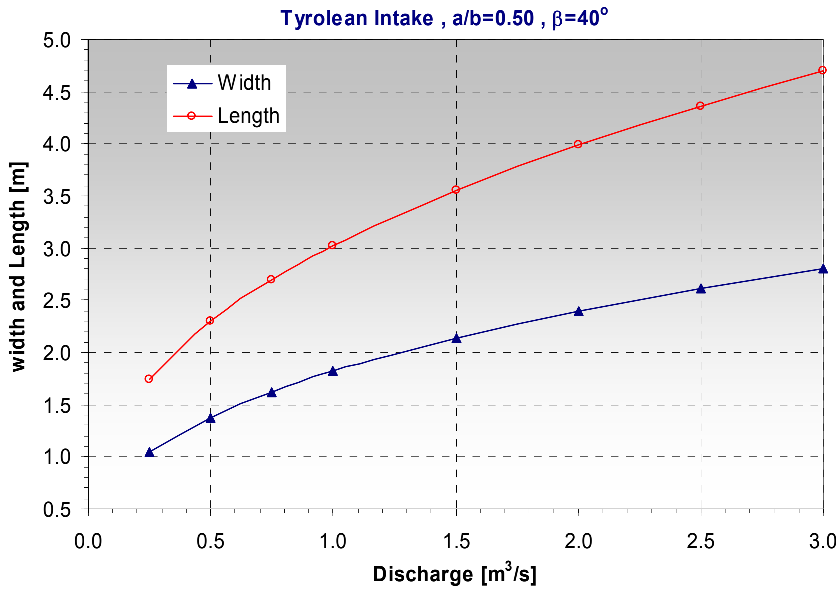


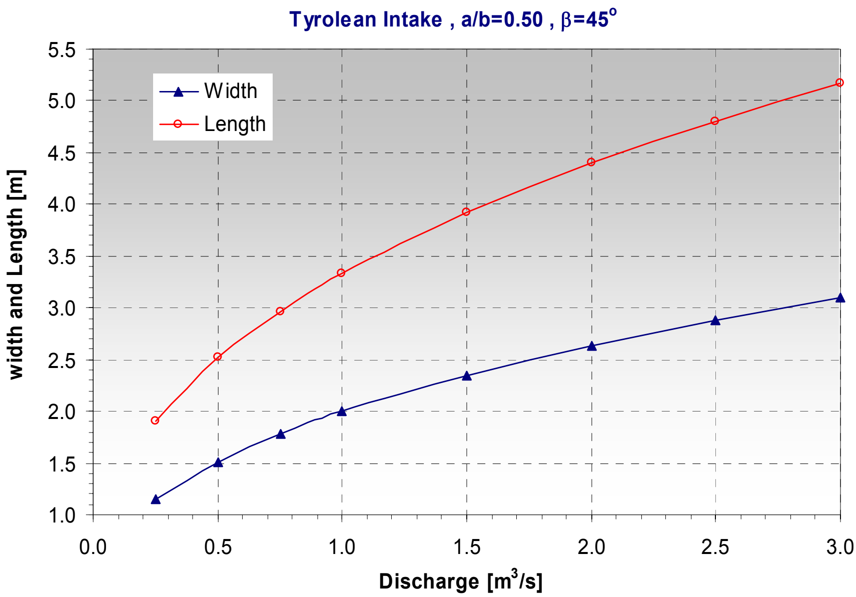
A2.1: Width and length of Tyrolean intake as a function of discharge, a/b and β



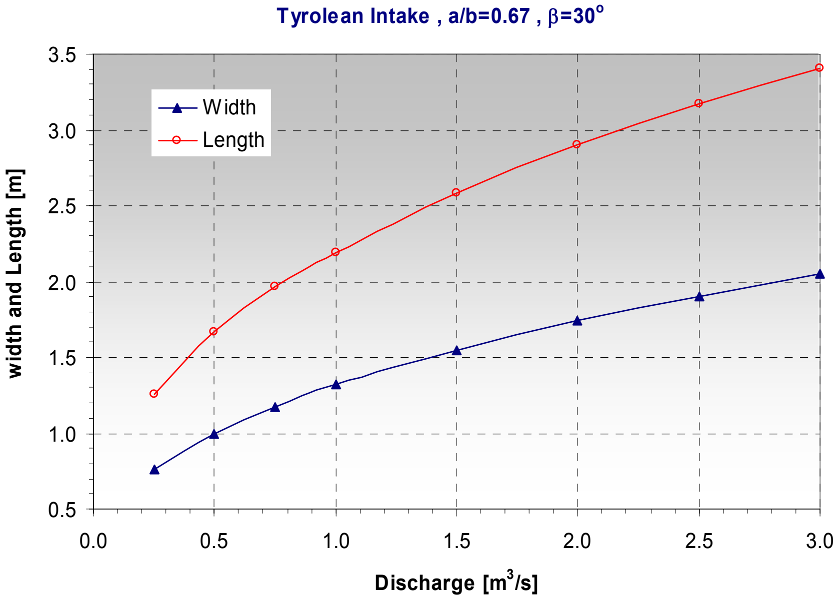


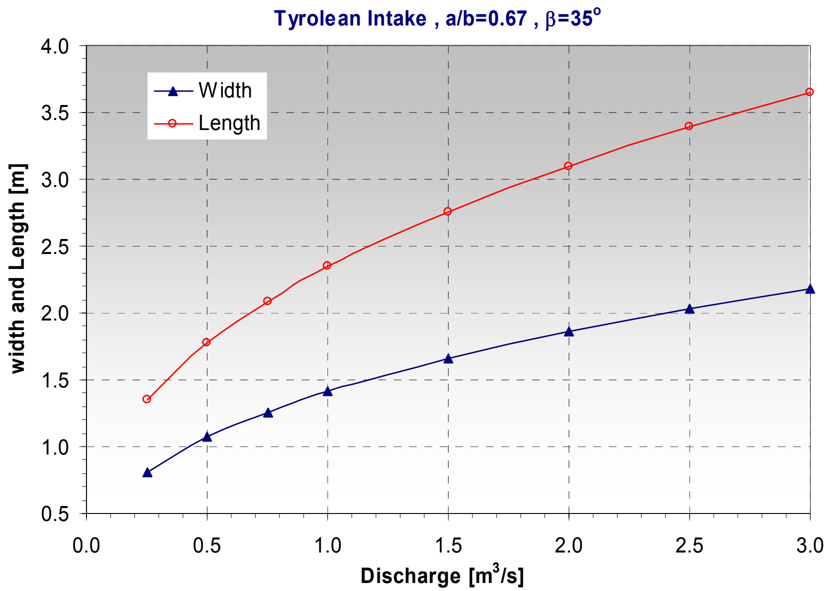
A2.1: Width and length of Tyrolean intake as a function of discharge, a/b and β



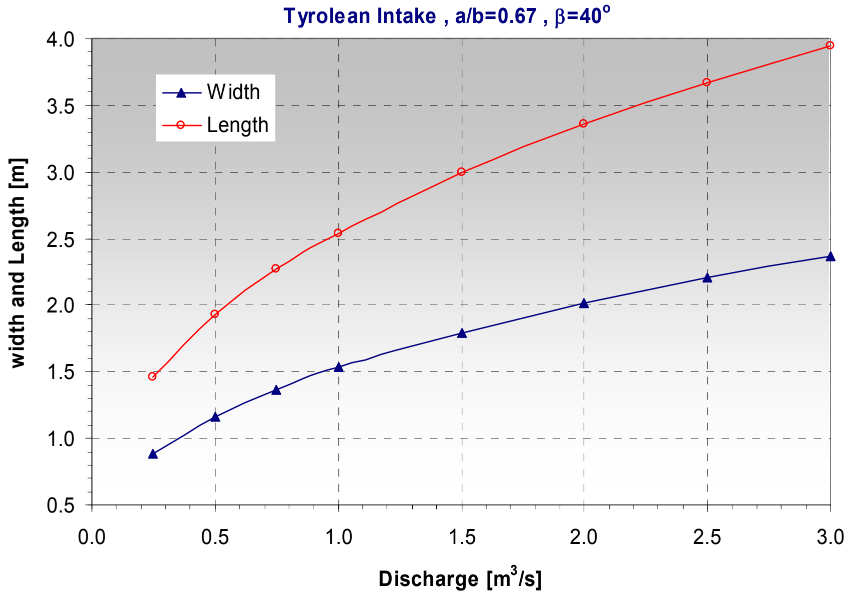


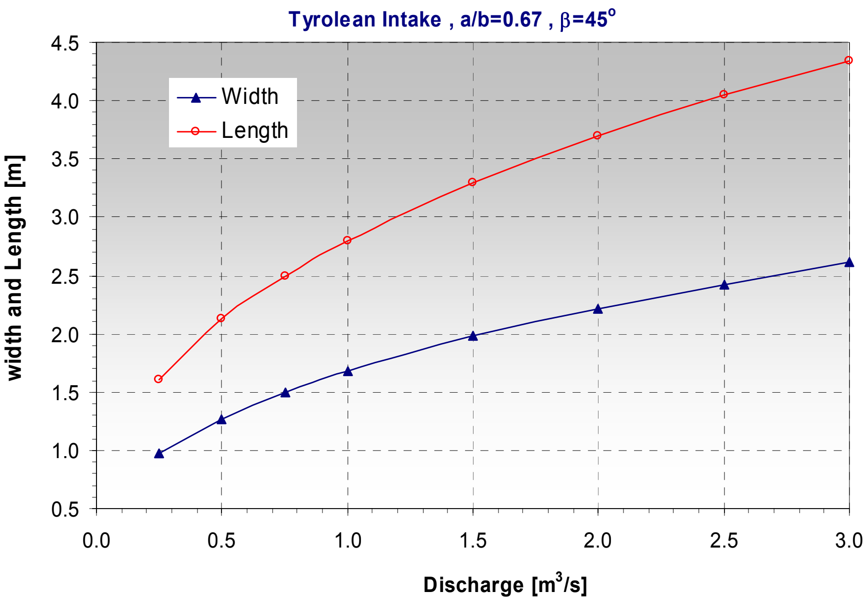
A2.1: Width and length of Tyrolean intake as a function of discharge, a/b and β



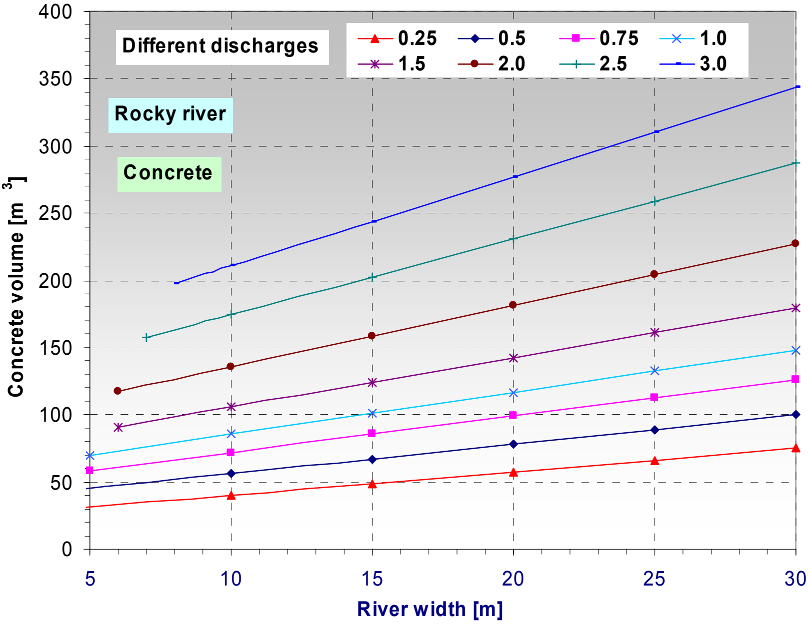


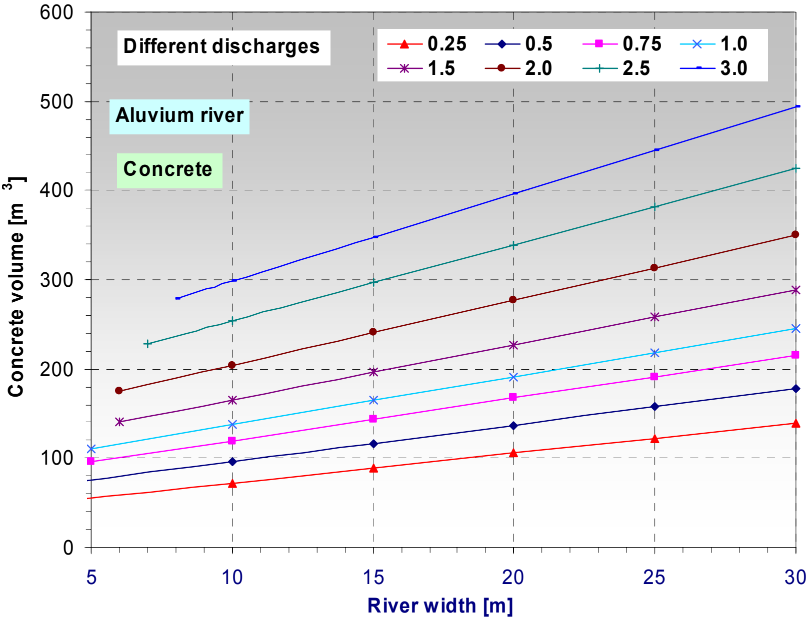
A2.1: Width and length of Tyrolean intake as a function of discharge, a/b and β



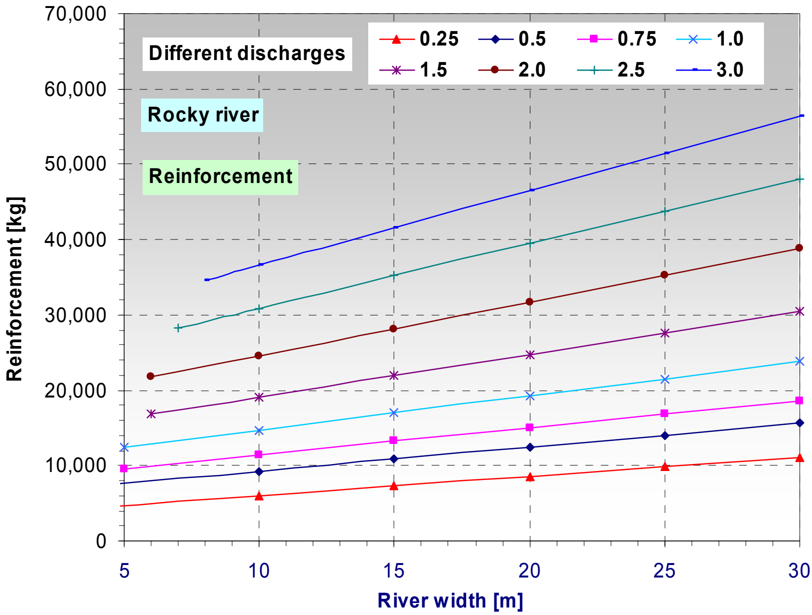


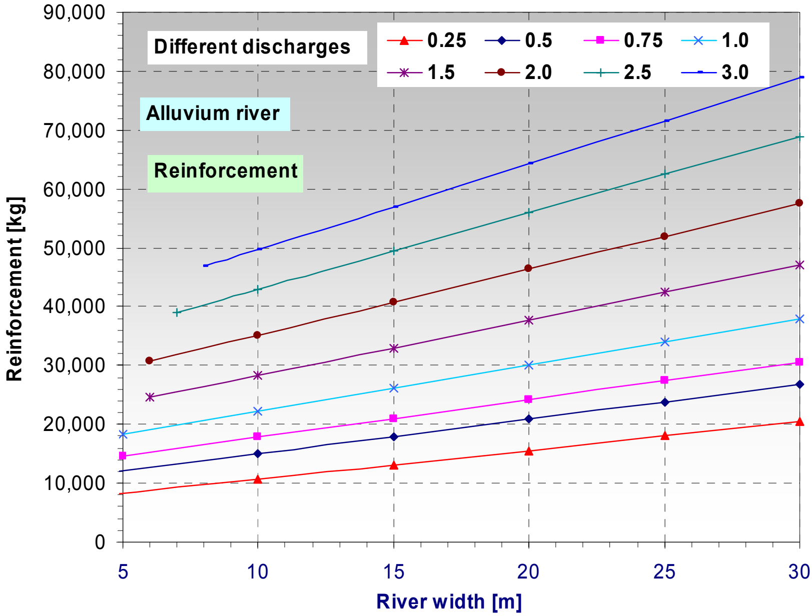
A2.2: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and β=30°



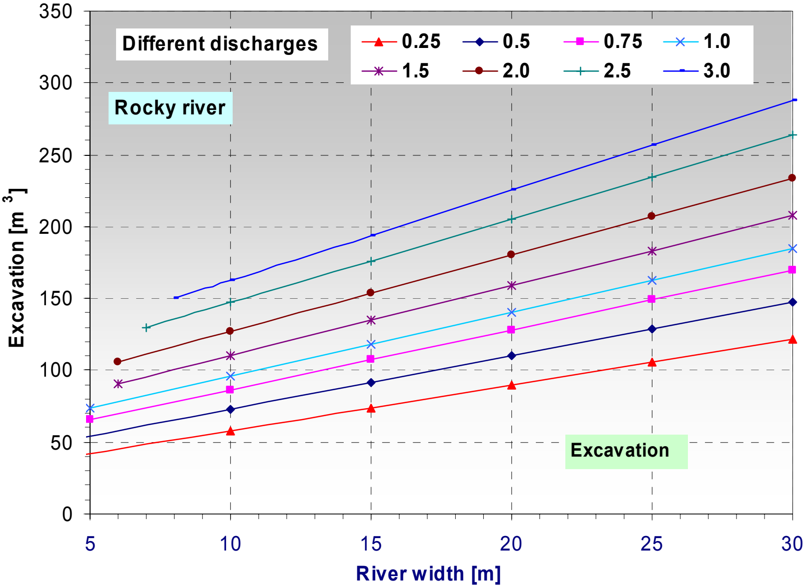


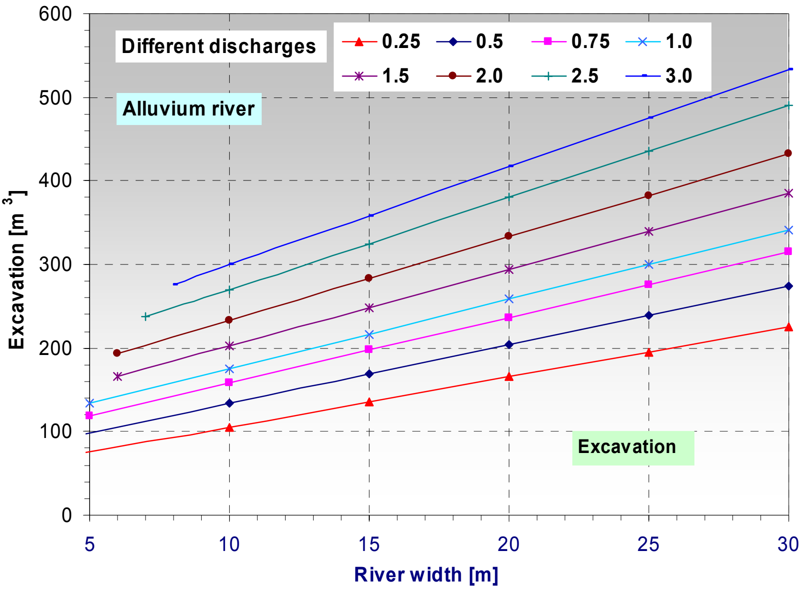
A2.2: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and β=30°



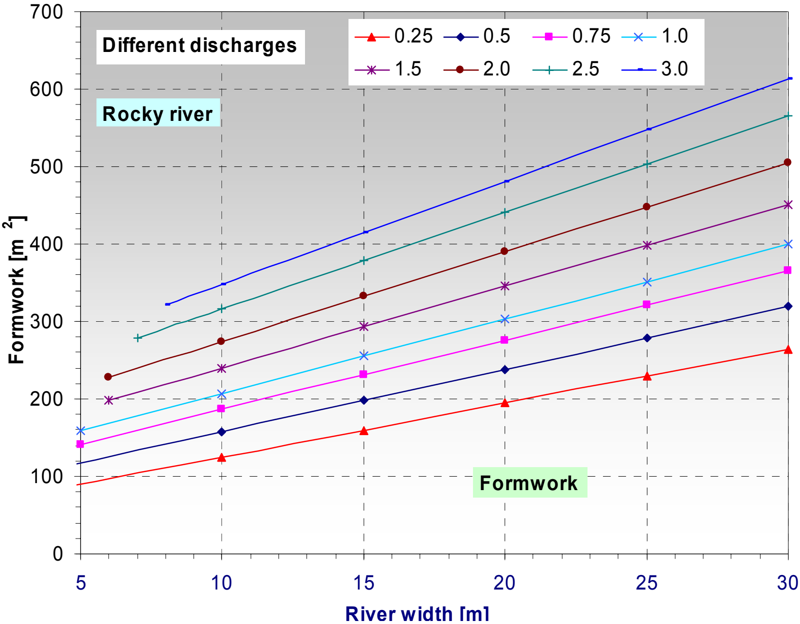


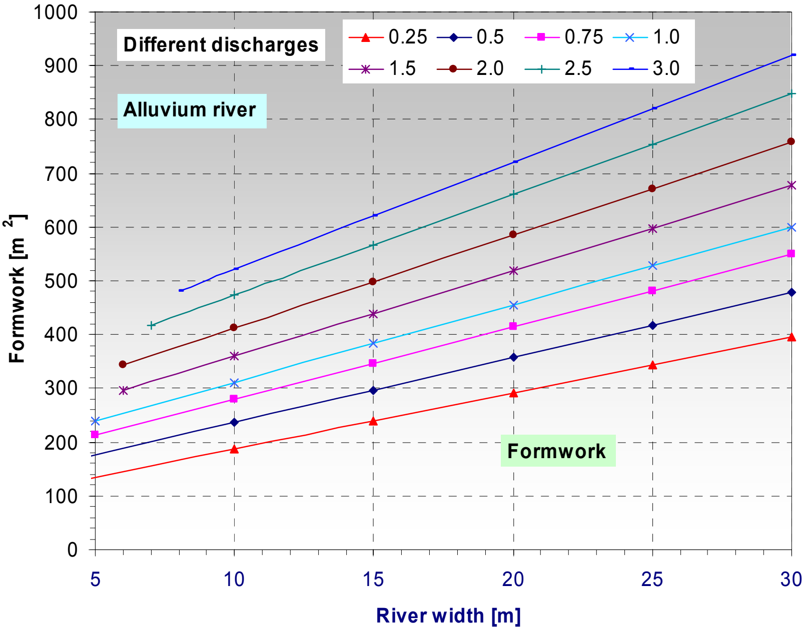
A2.2: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and β=30°



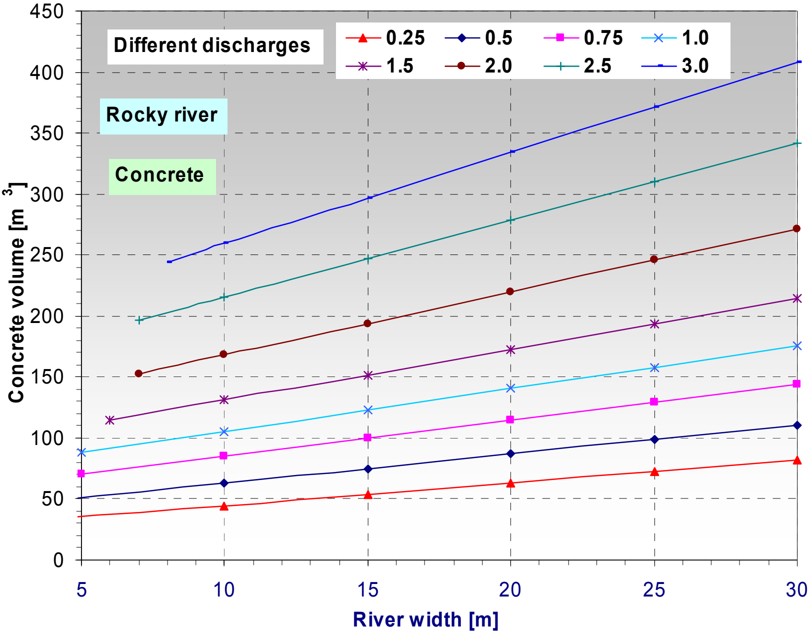


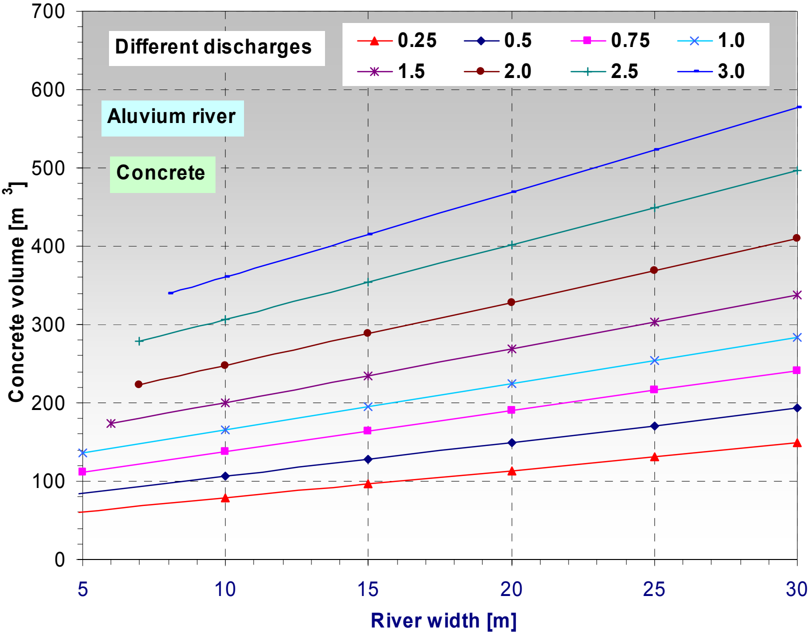
A2.2: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and β=30°



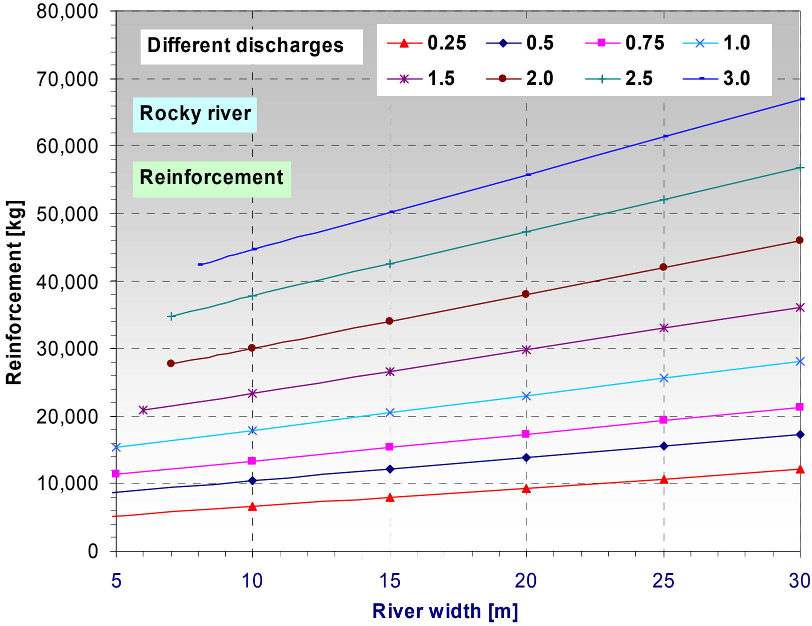


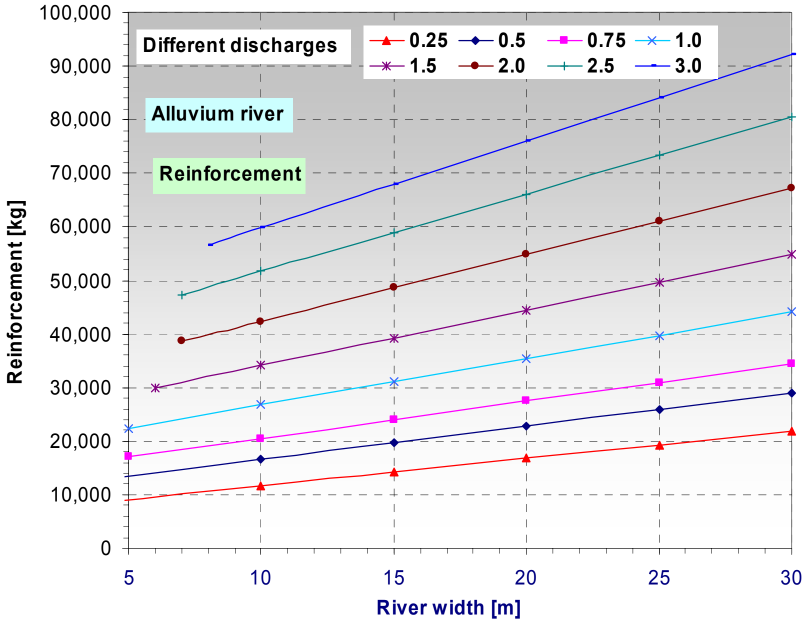
A2.3: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and $\beta=35^o$



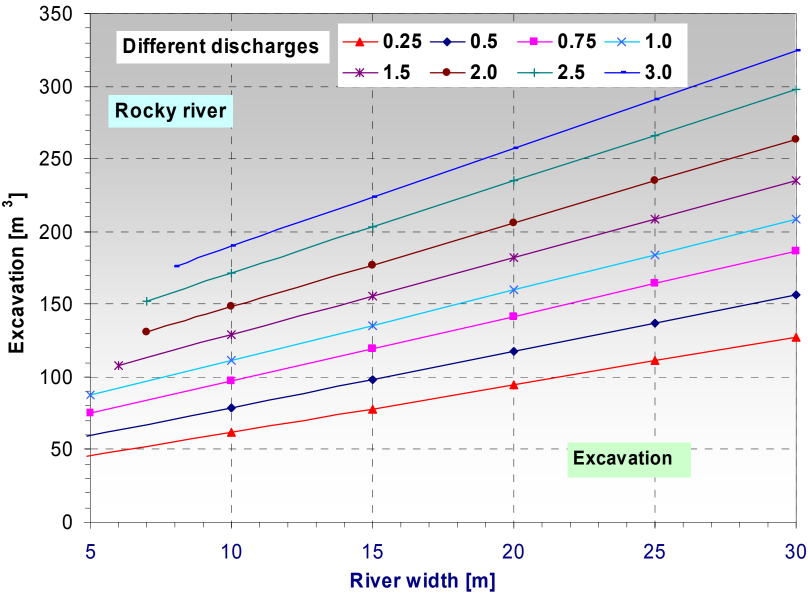


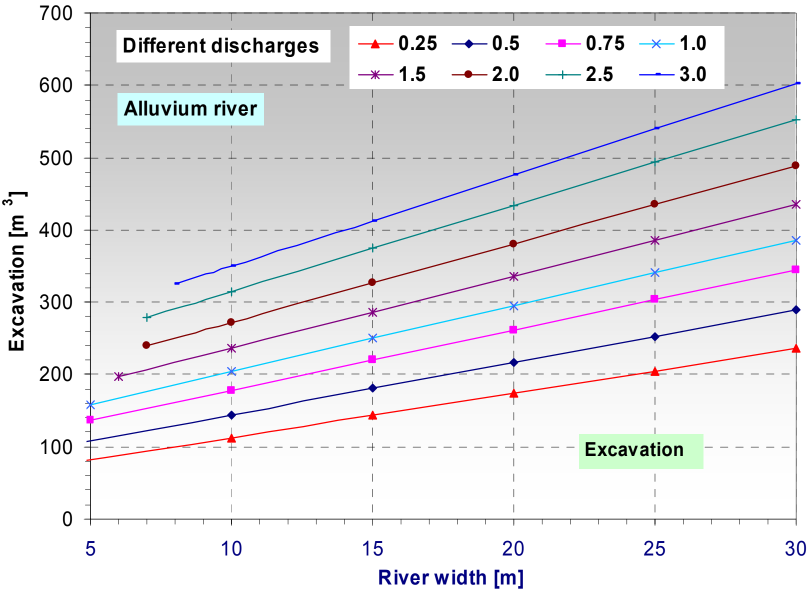
A2.3: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and $\beta=35^o$



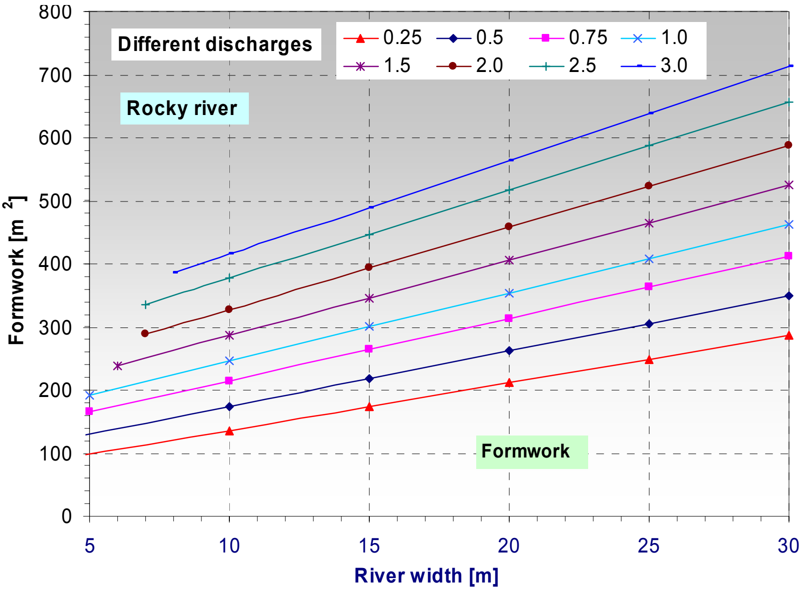


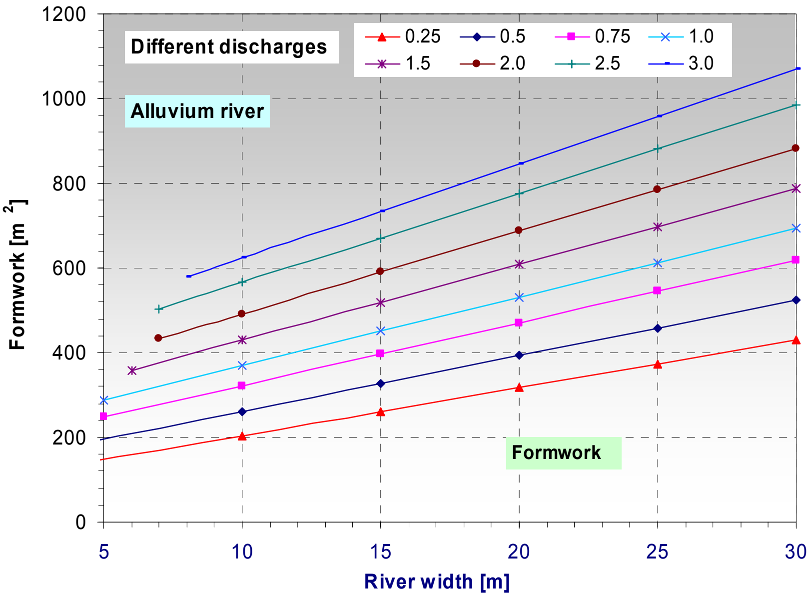
A2.3: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and $\beta=35^o$



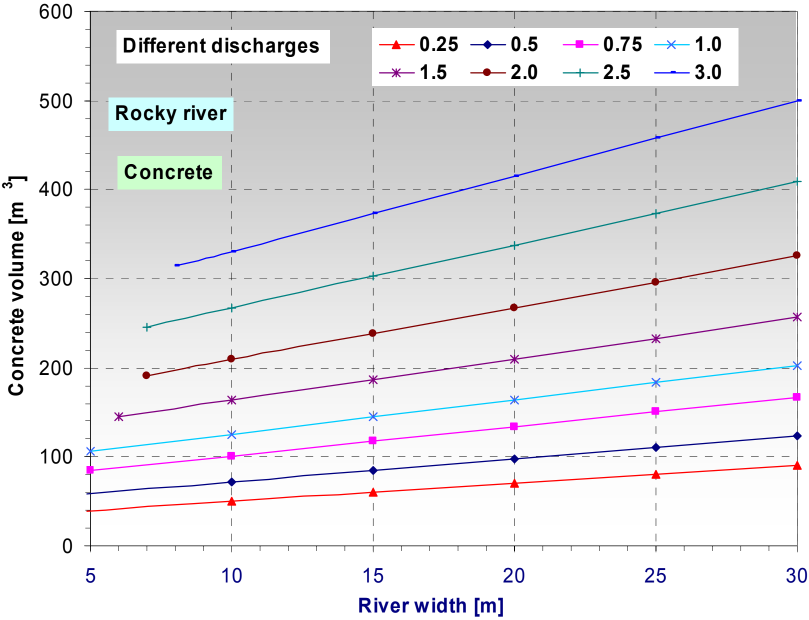


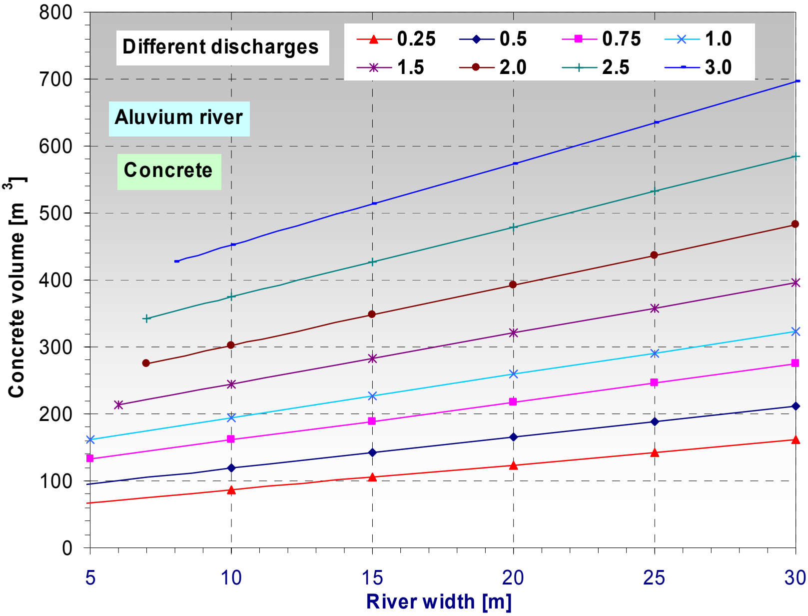
A2.3: Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and $\beta=35^o$



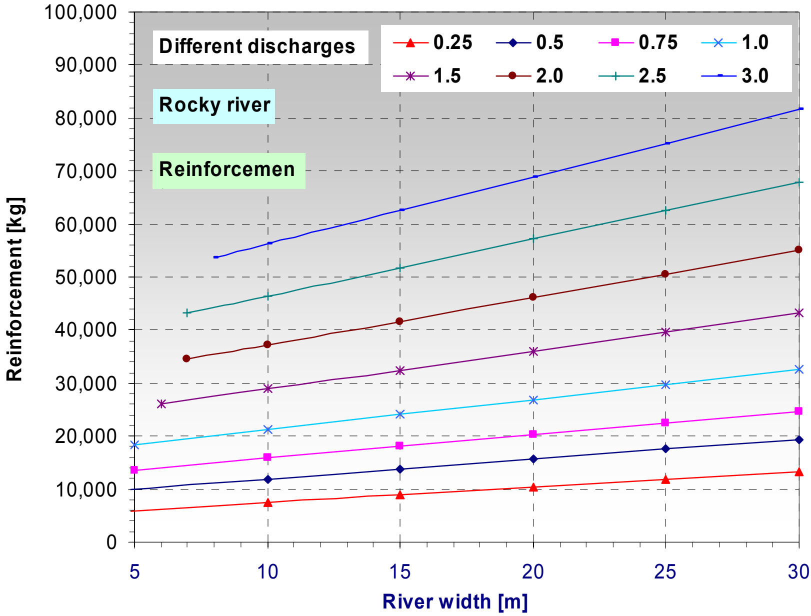


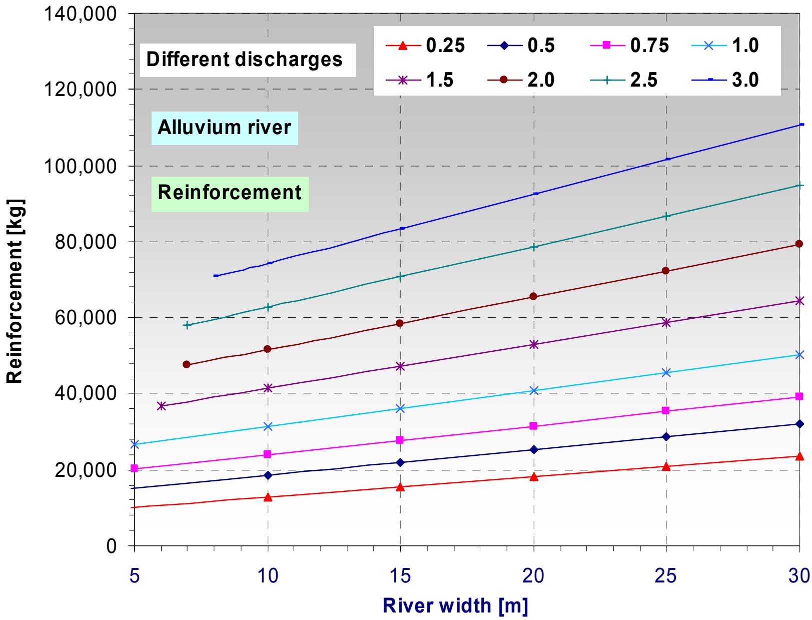
A2.4 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and β=40°



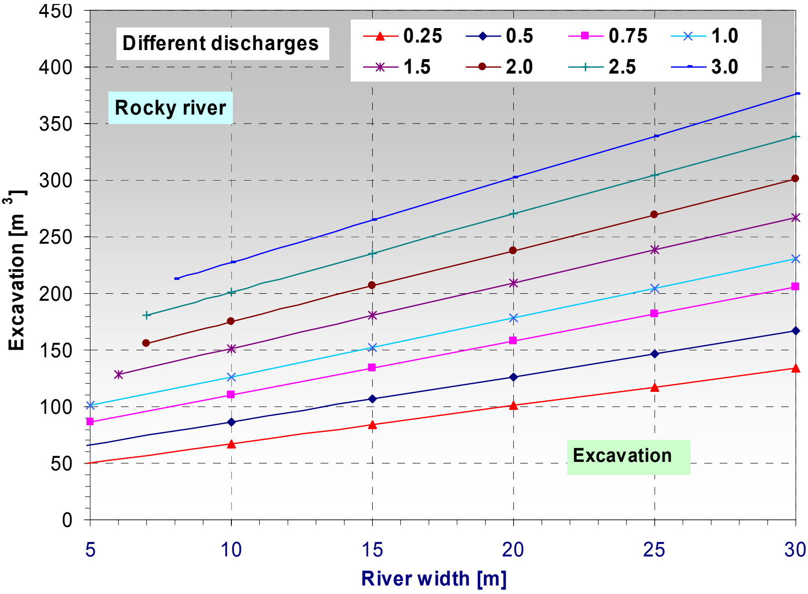


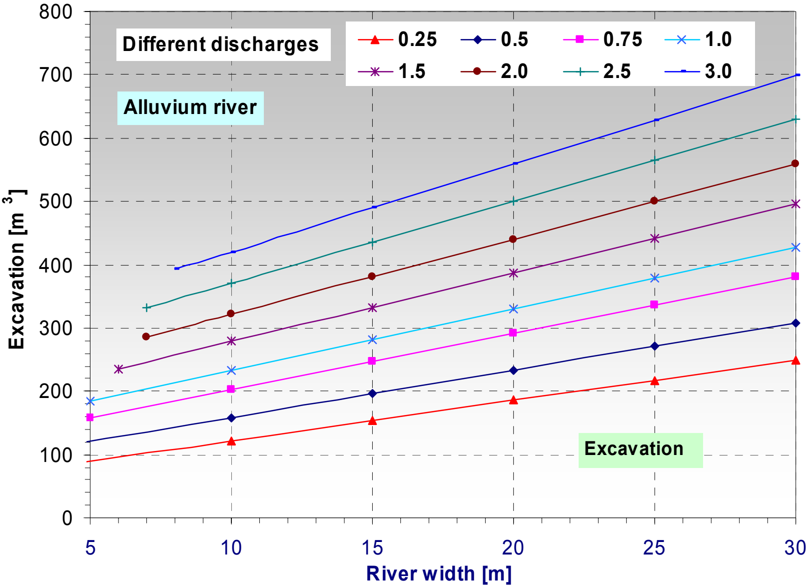
A2.4 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and β=40°



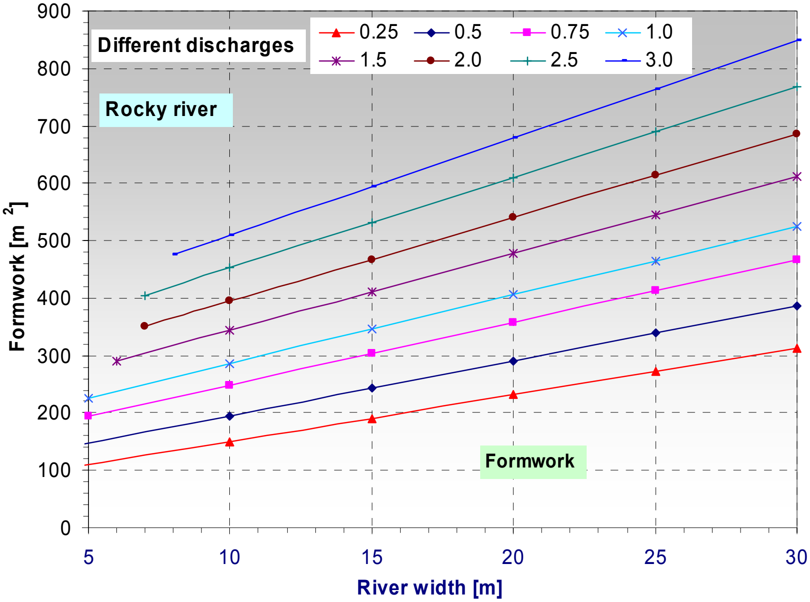


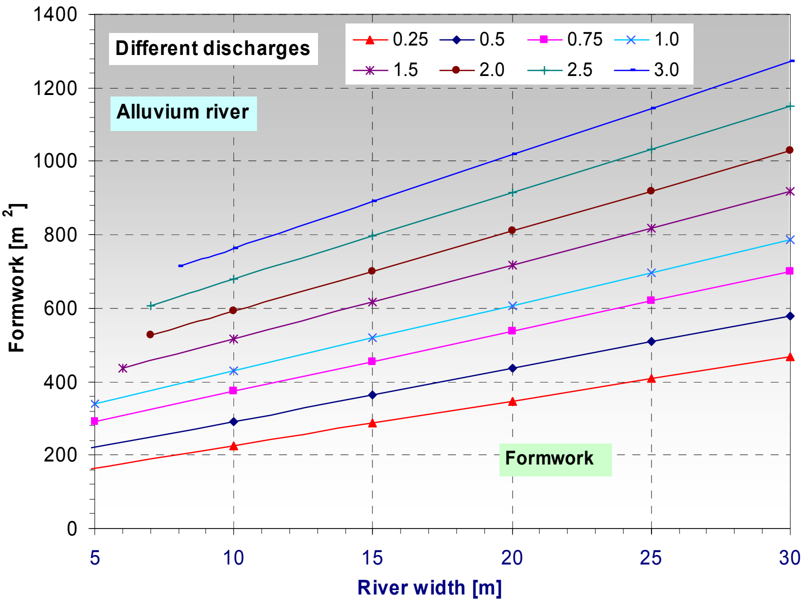
A2.4 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and β=40°



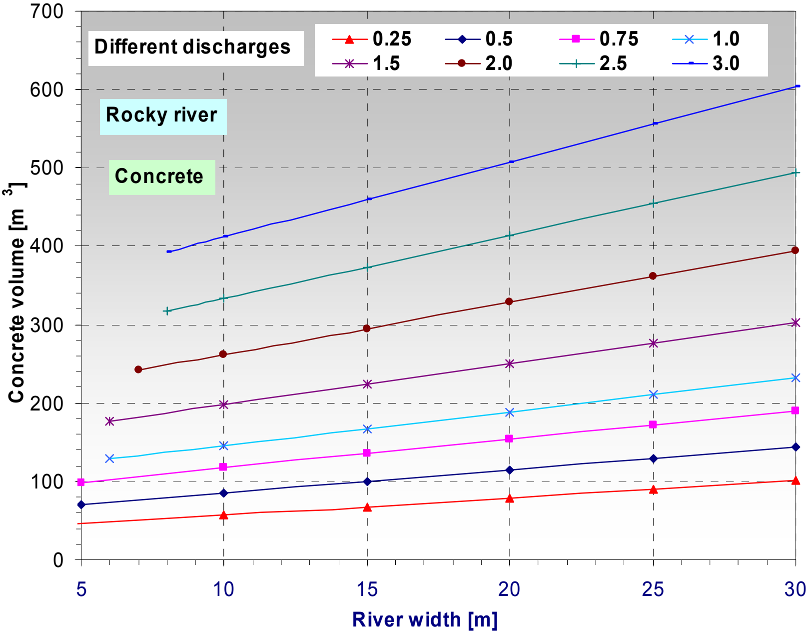


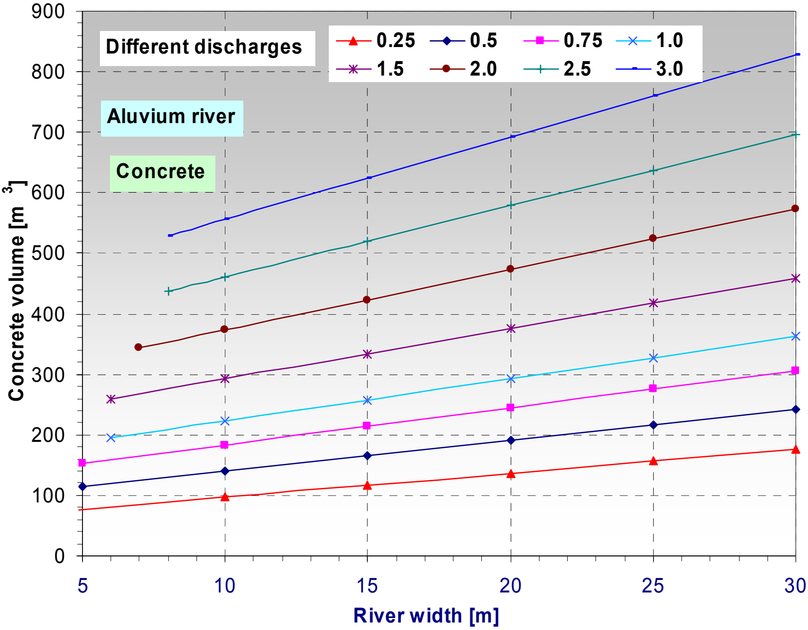
A2.4 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and β=40°



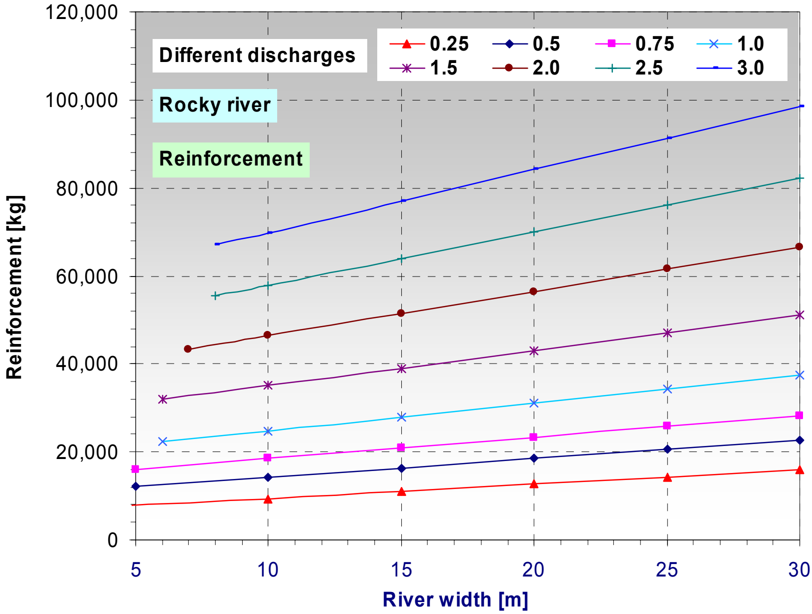


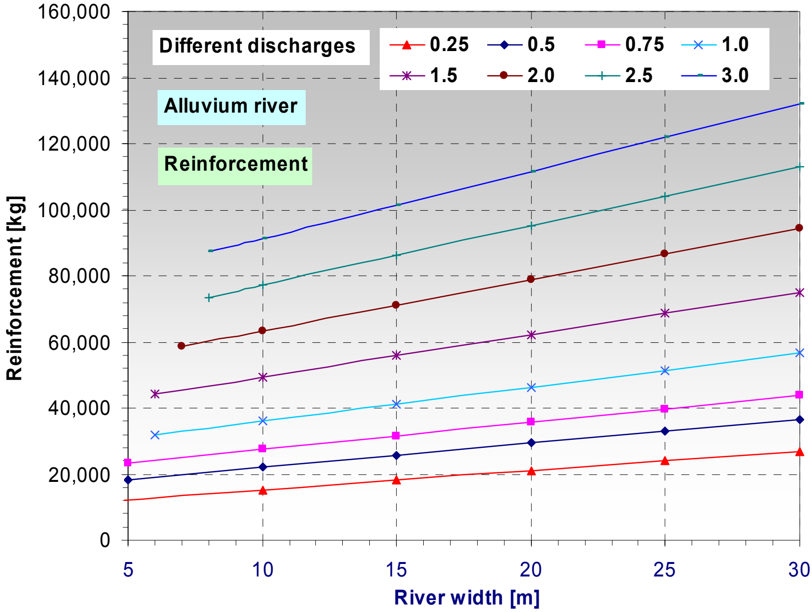
A2.5 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and $\beta=45^o$



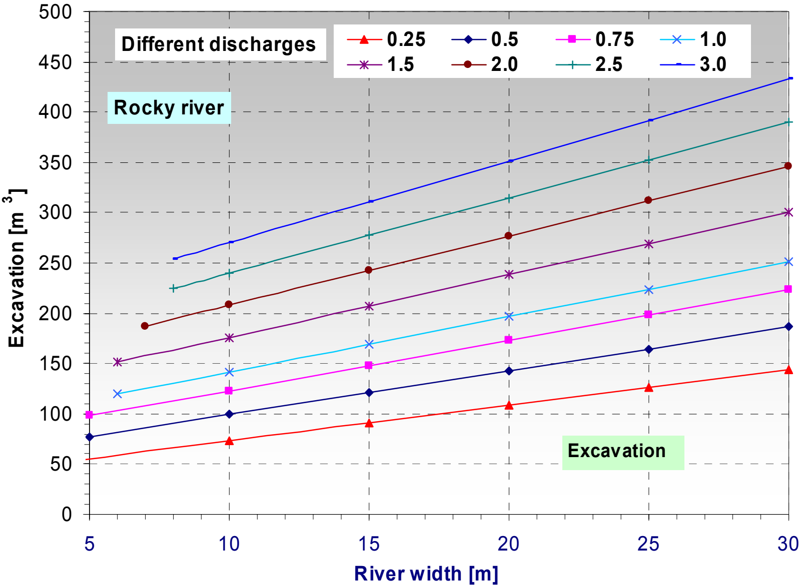


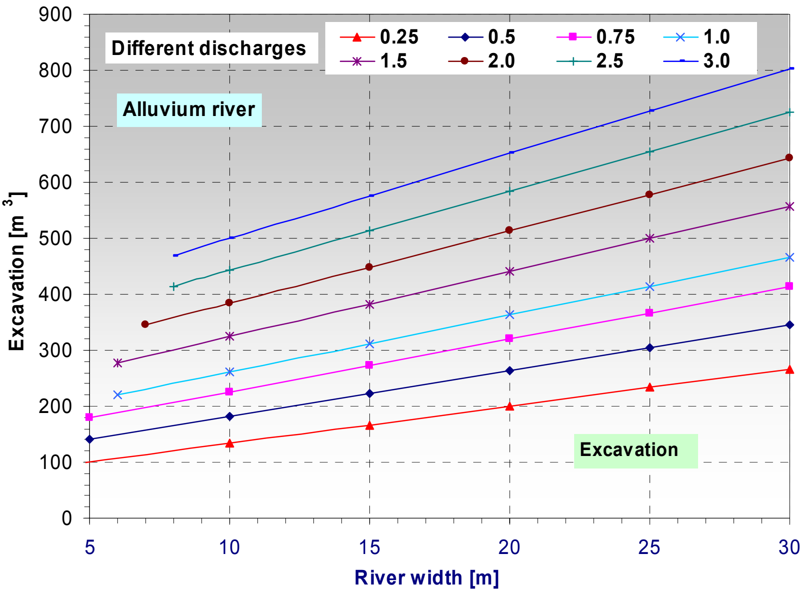
A2.5 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and β=45°



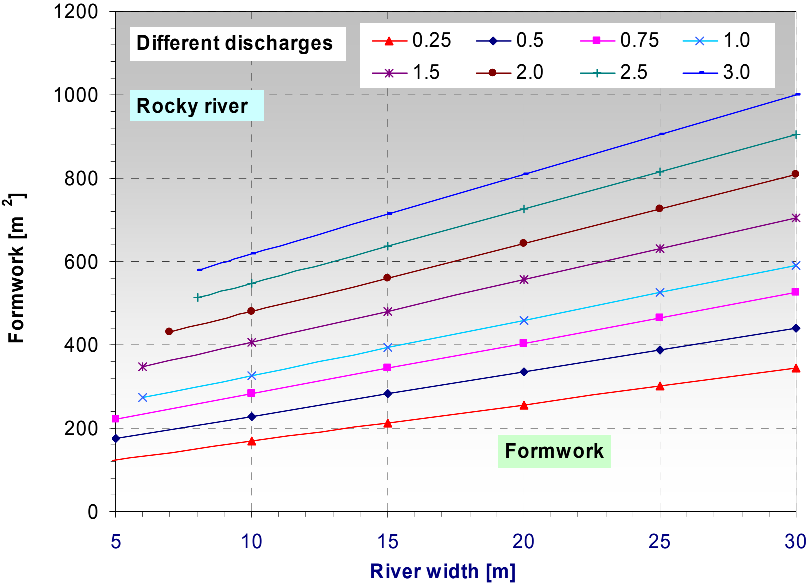


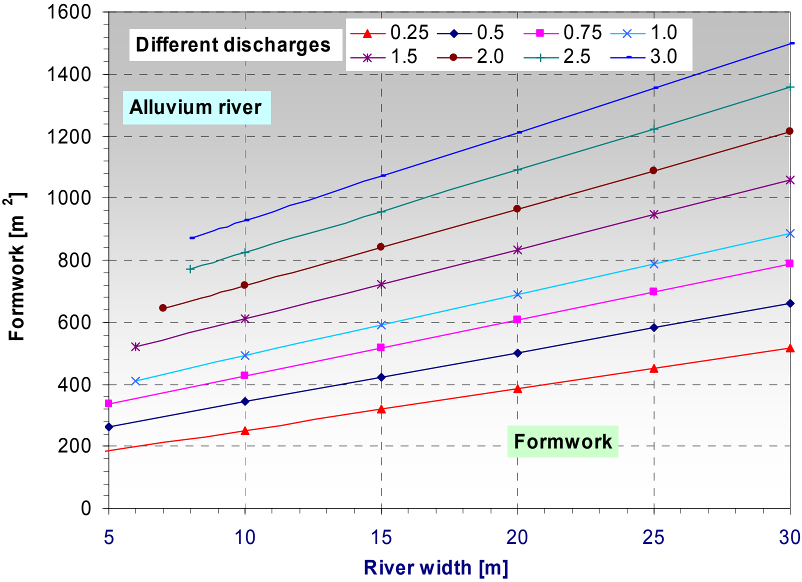
A2.5 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and $\beta=45^o$



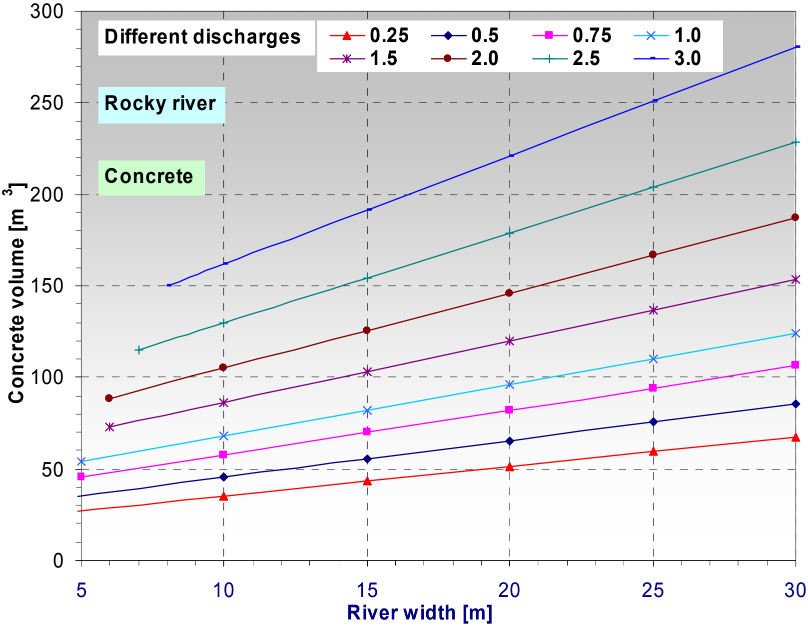


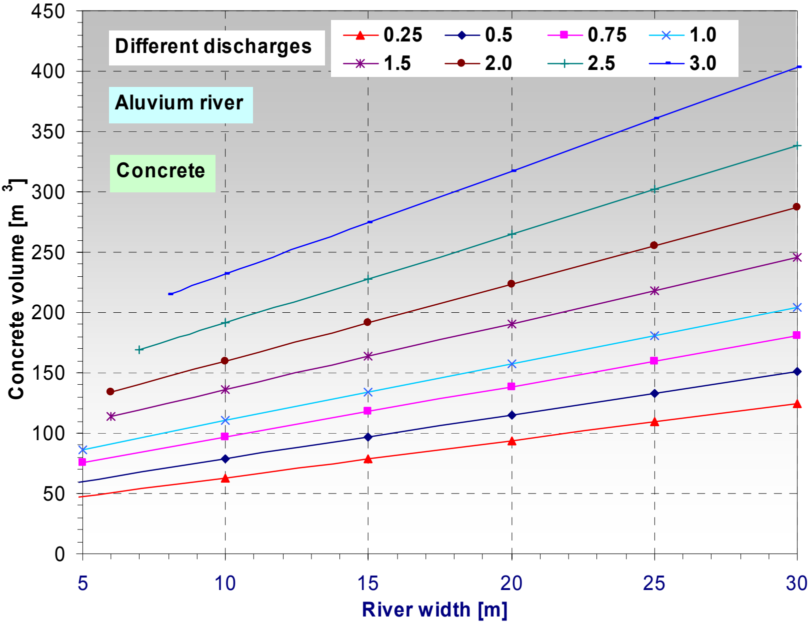
A2.5 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/3 and β=45°



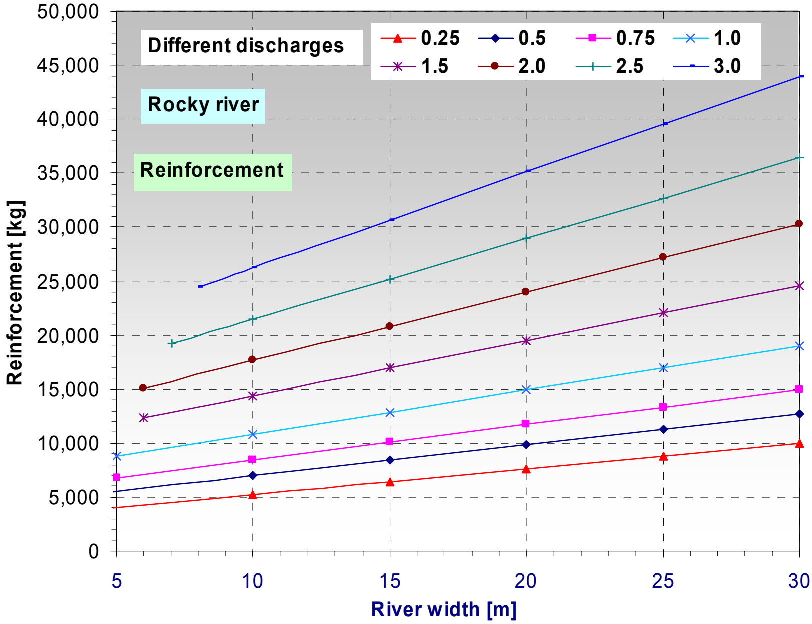


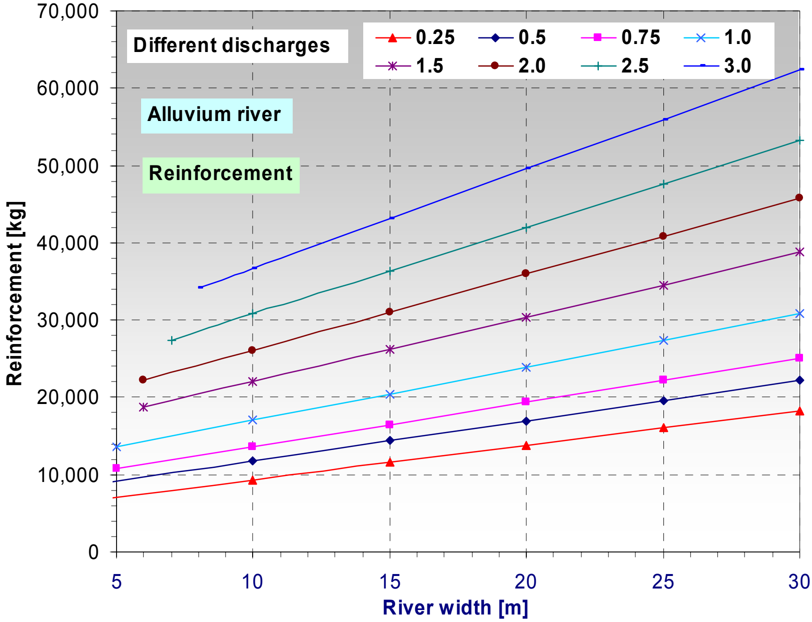
A2.6 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and $\beta=30^o$



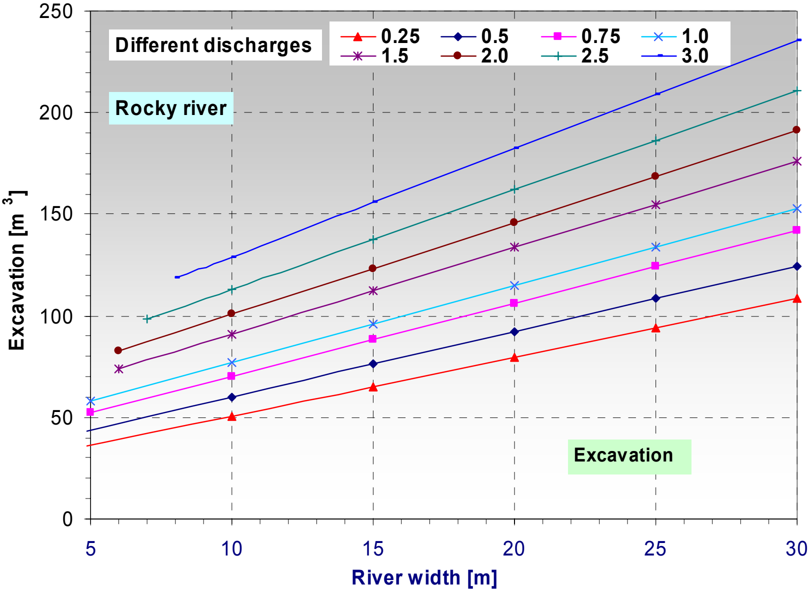


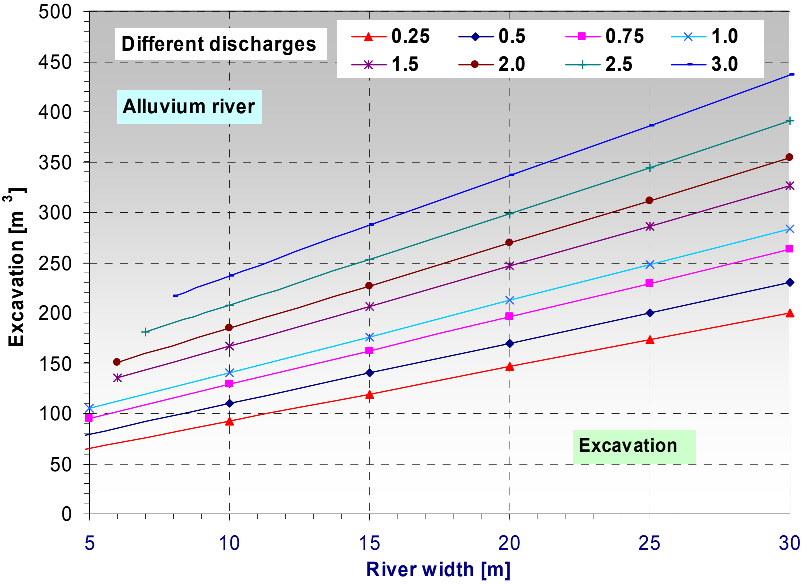
A2.6 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=30°



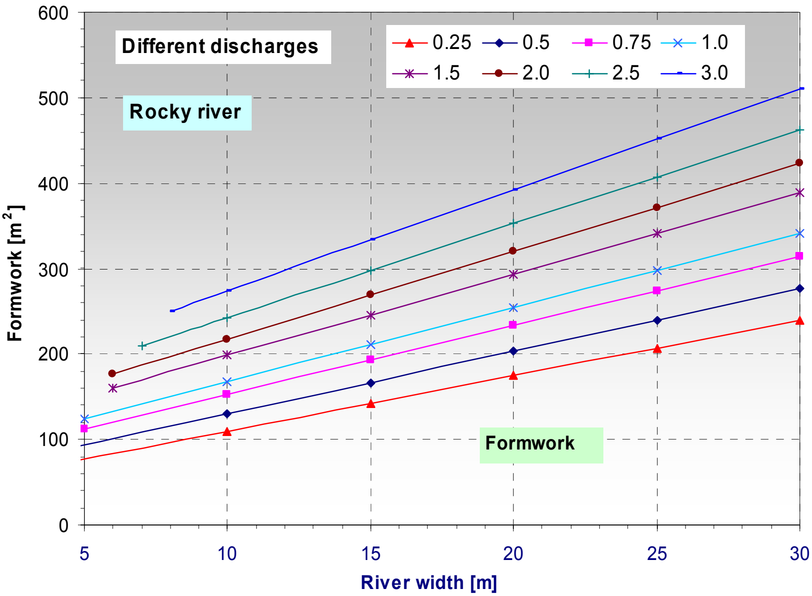


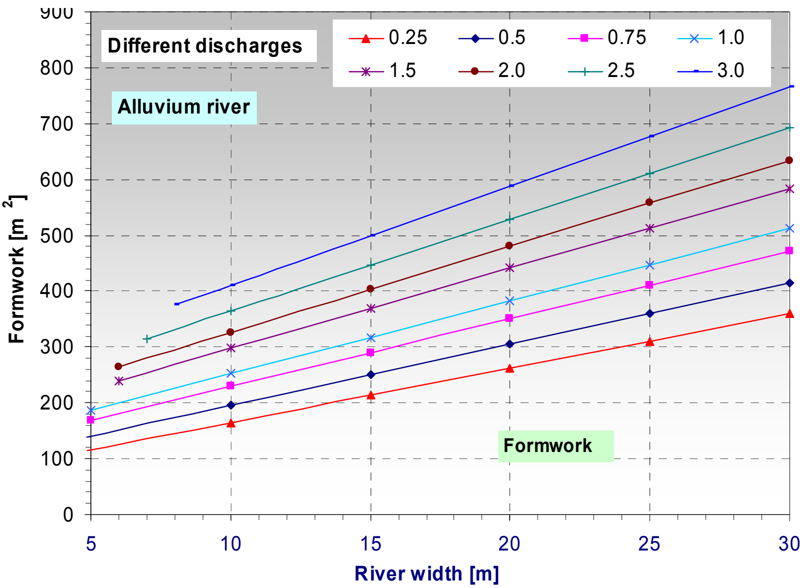
A2.6 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=30°



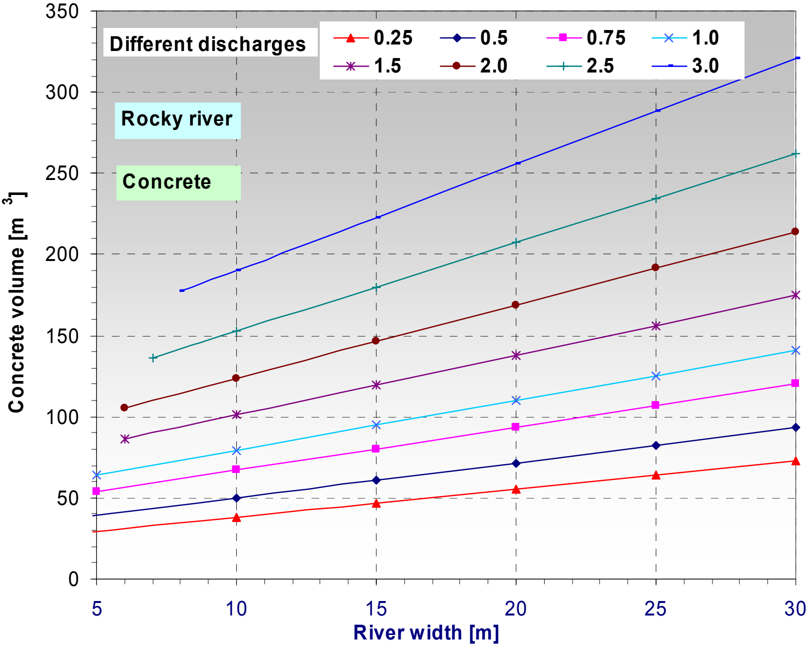


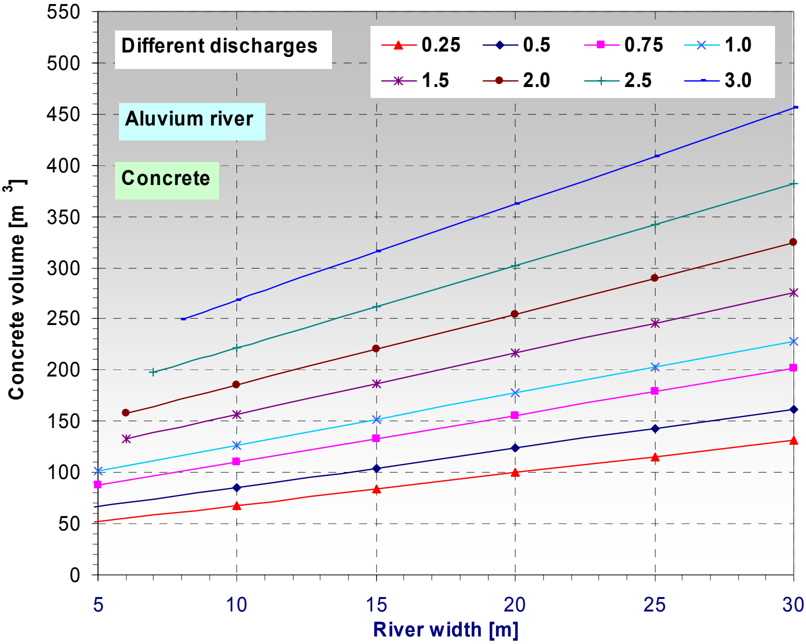
A2.6 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=30°



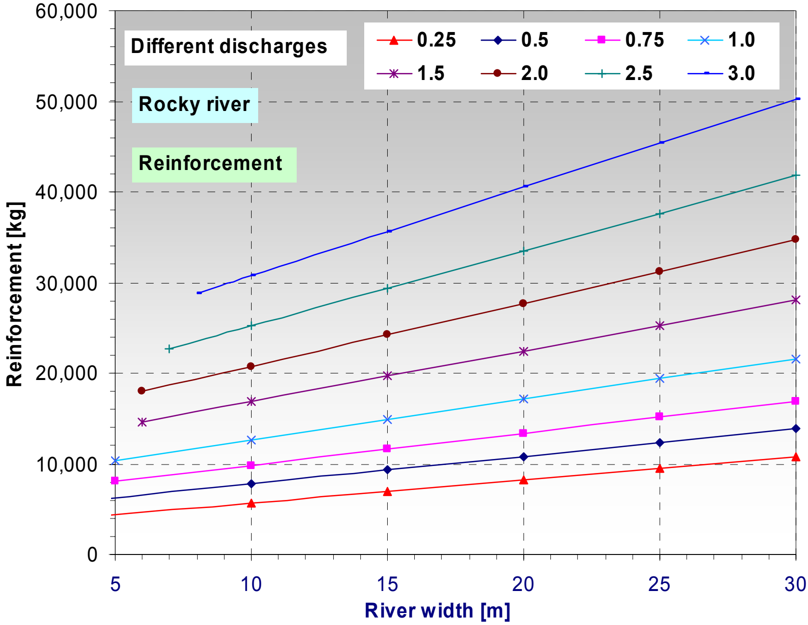


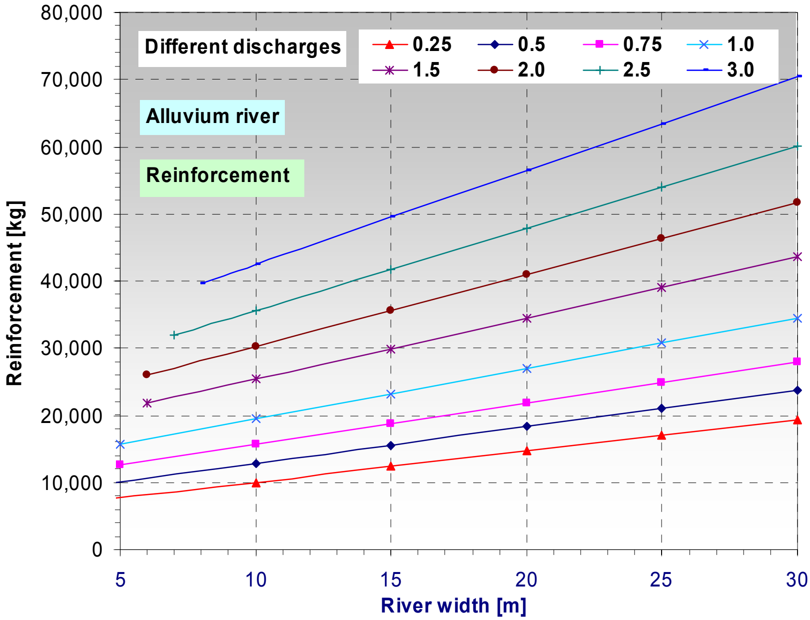
A2.7 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=35°



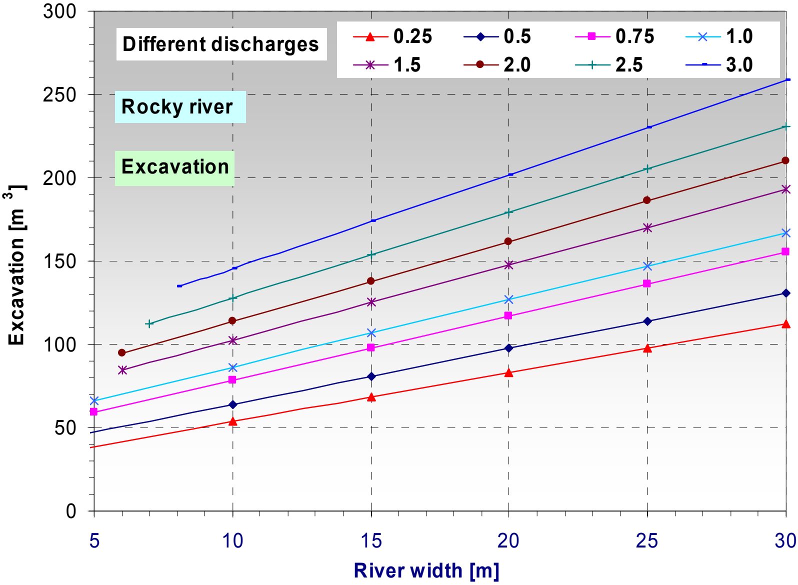


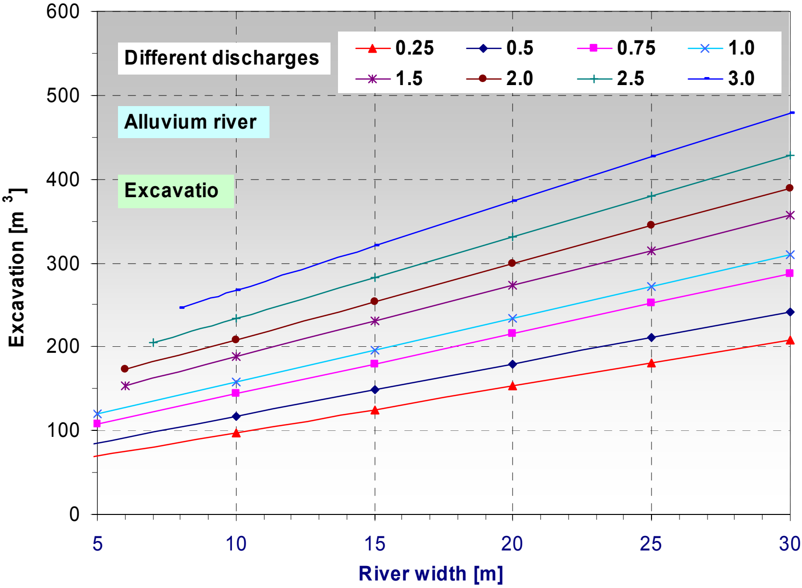
A2.7 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=35°



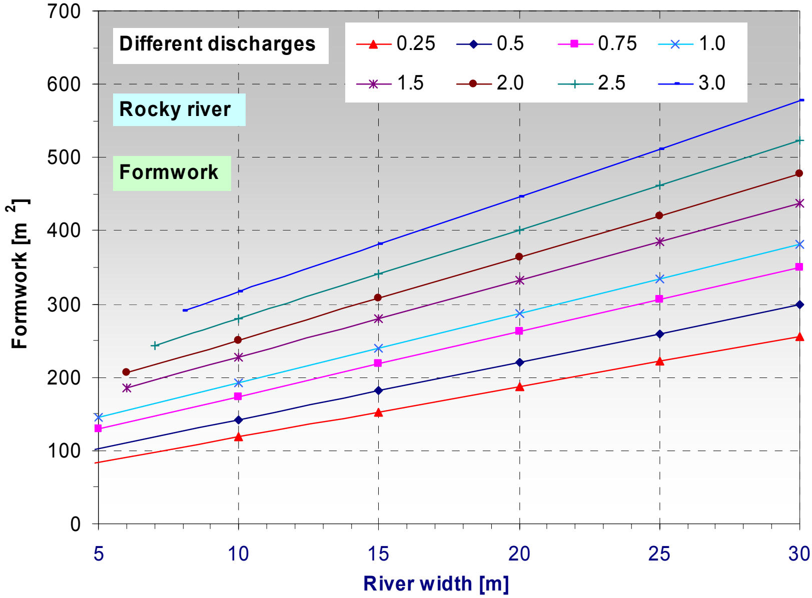


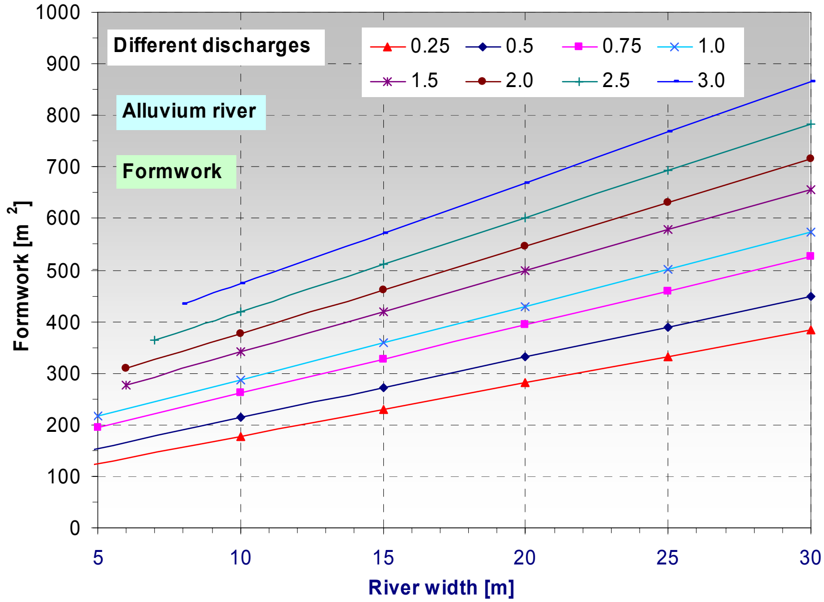
A2.7 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=35°



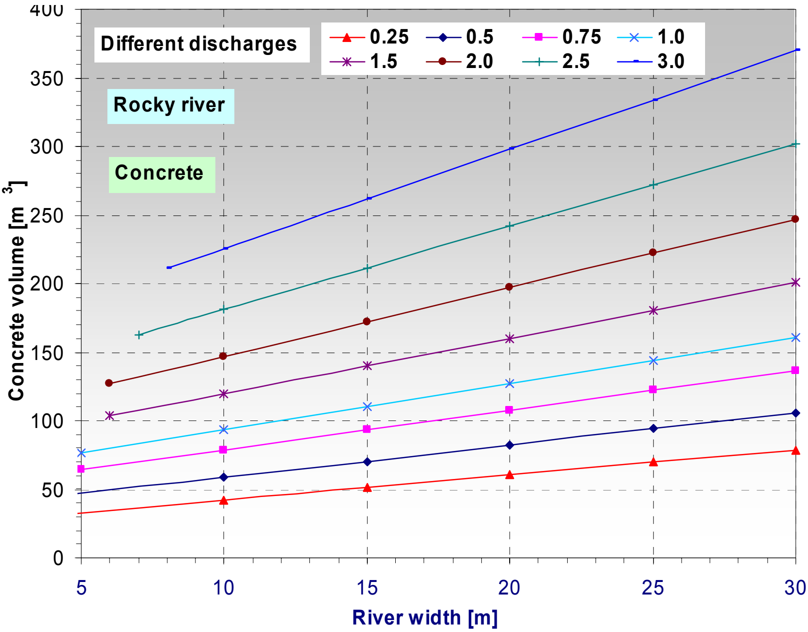


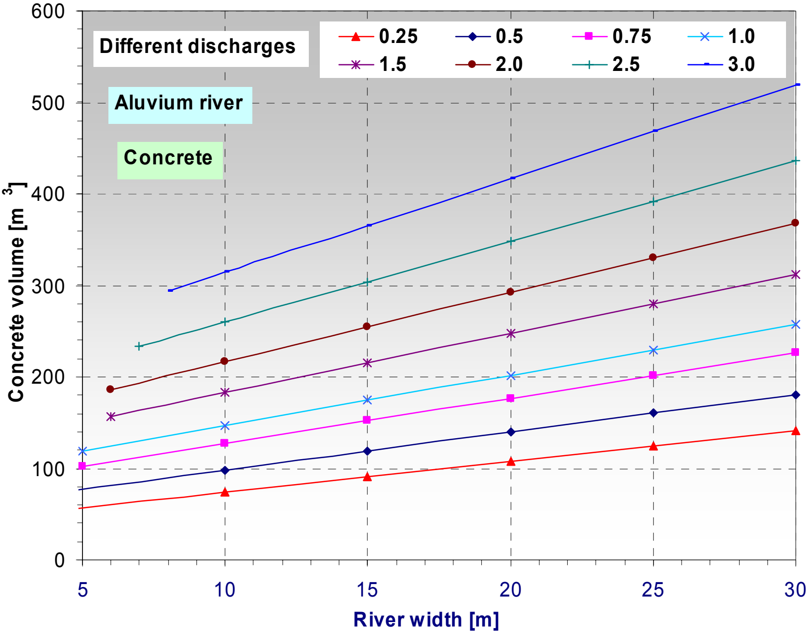
A2.7 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=35°



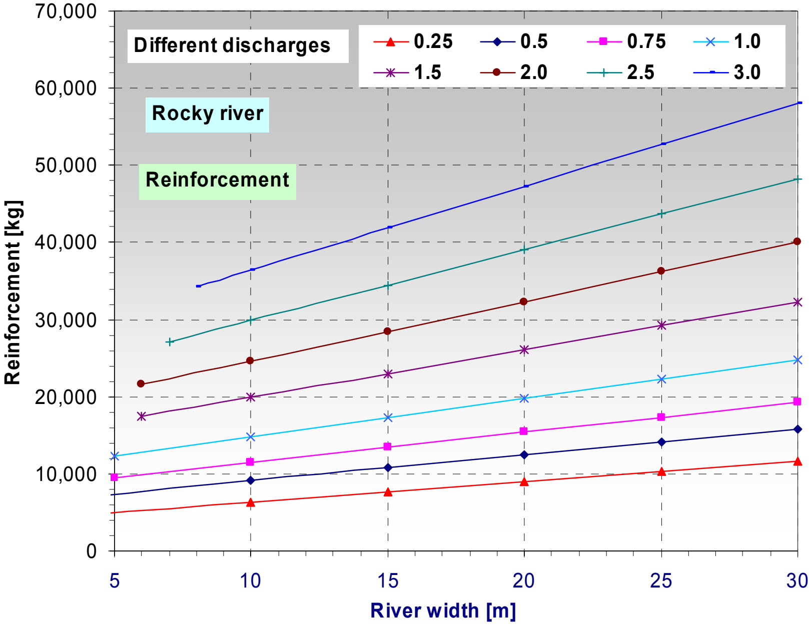


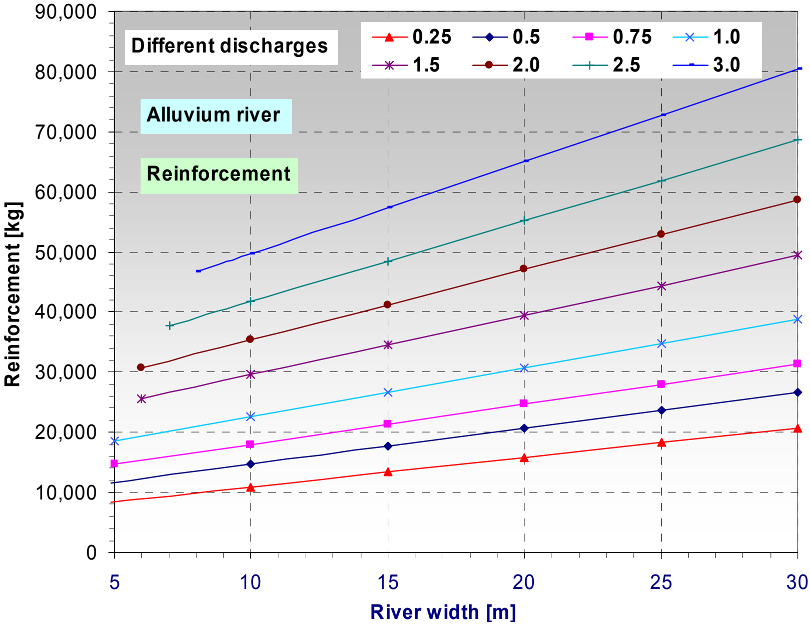
A2.8 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=40°



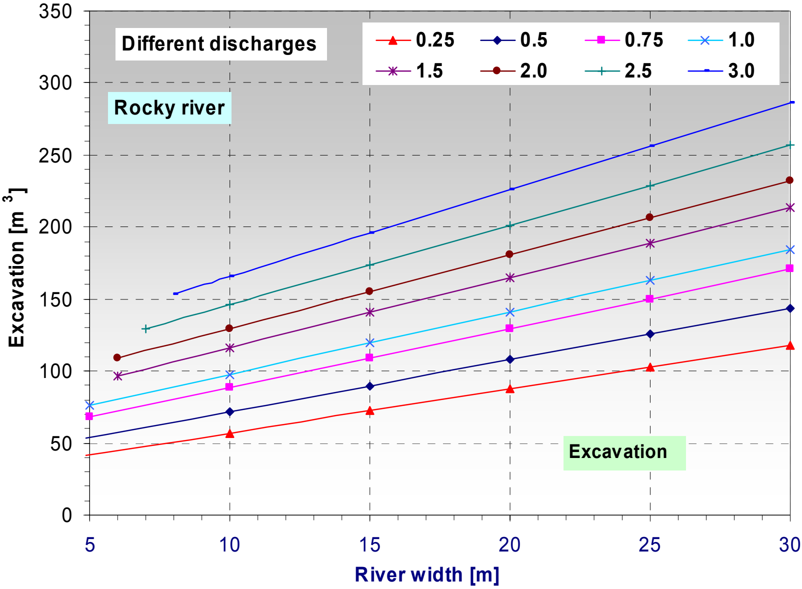


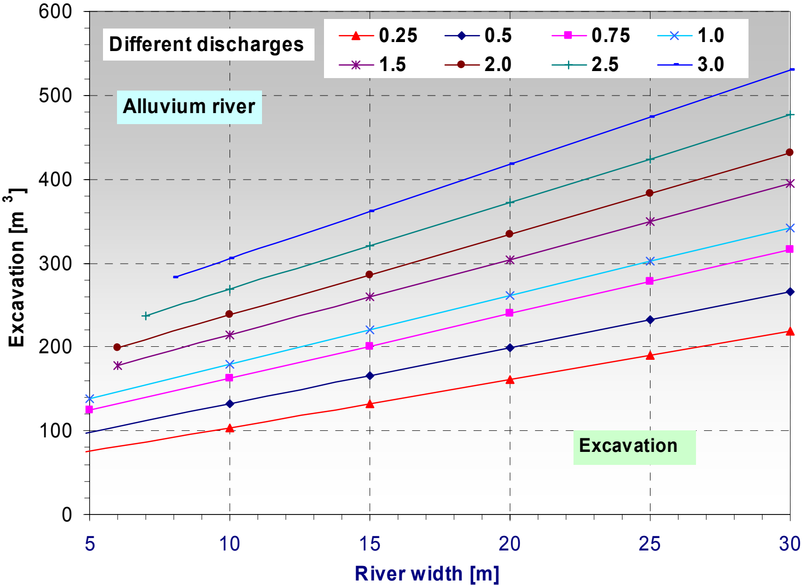
A2.8 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=40°



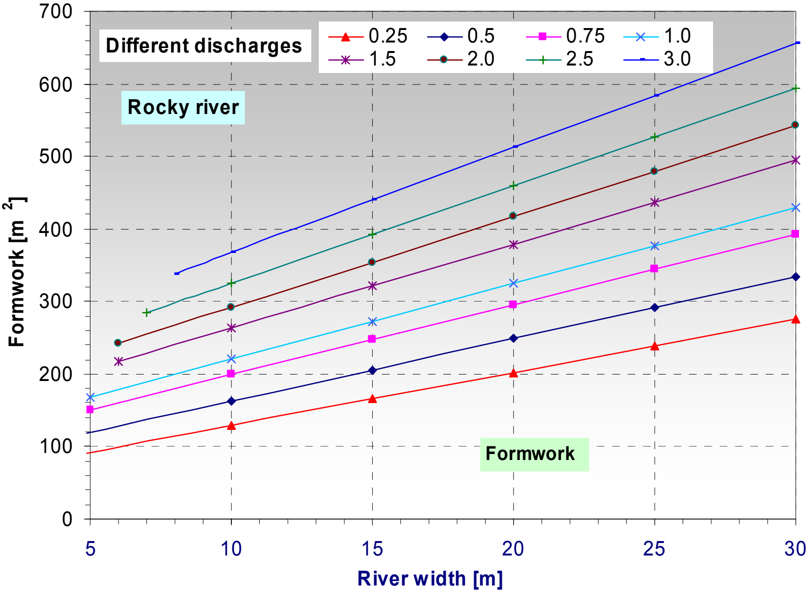


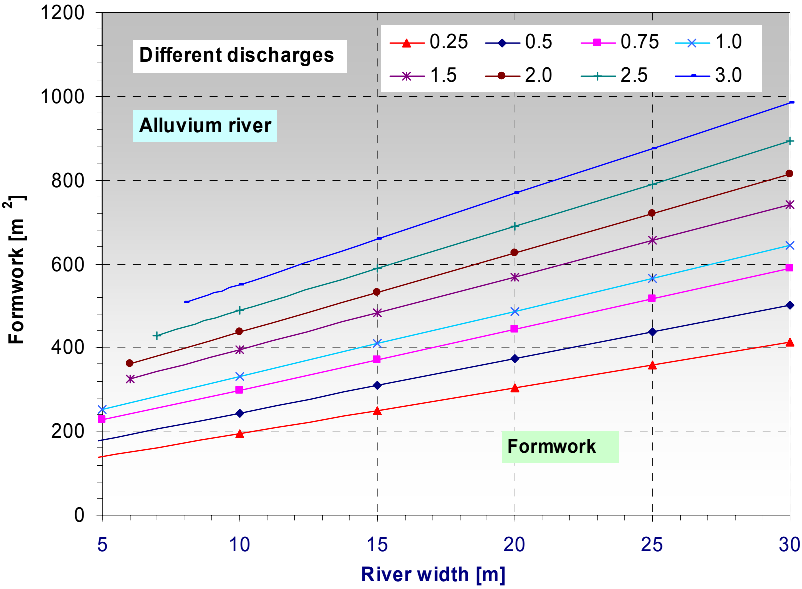
A2.8 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=40°



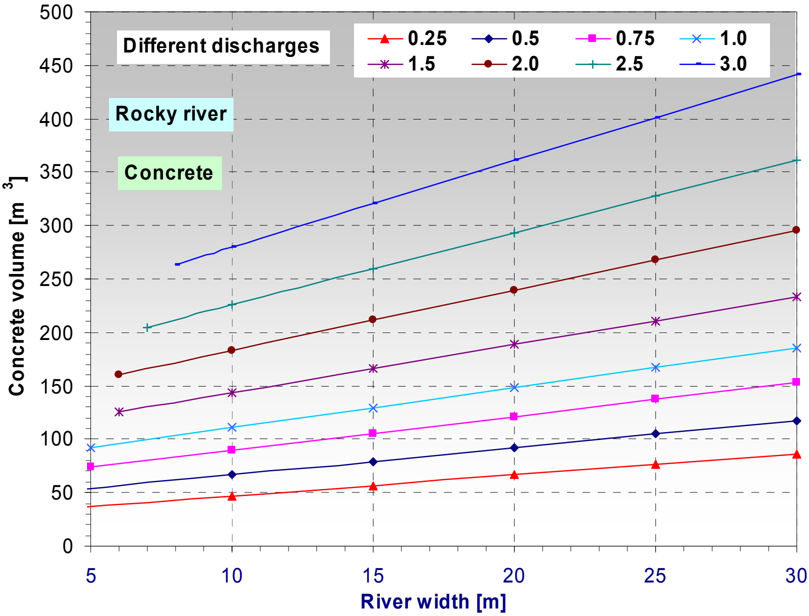


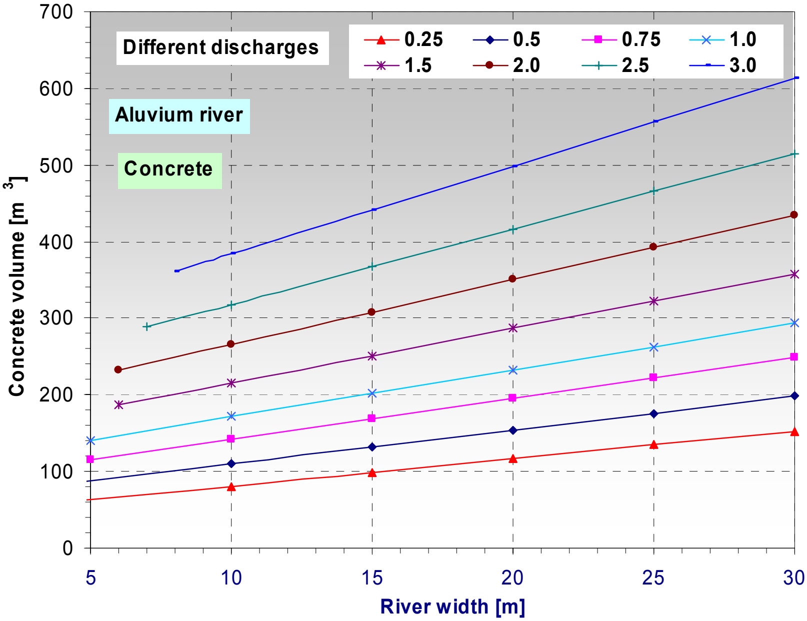
A2.8 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=40°



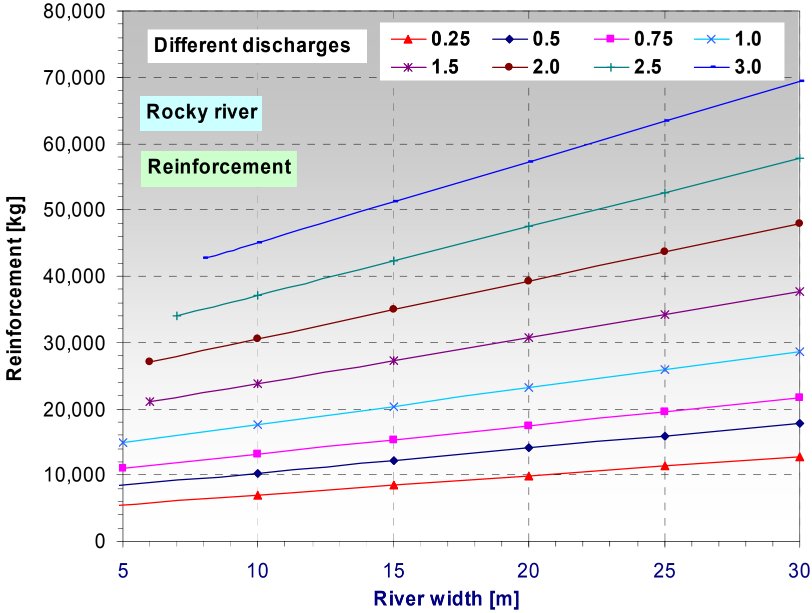


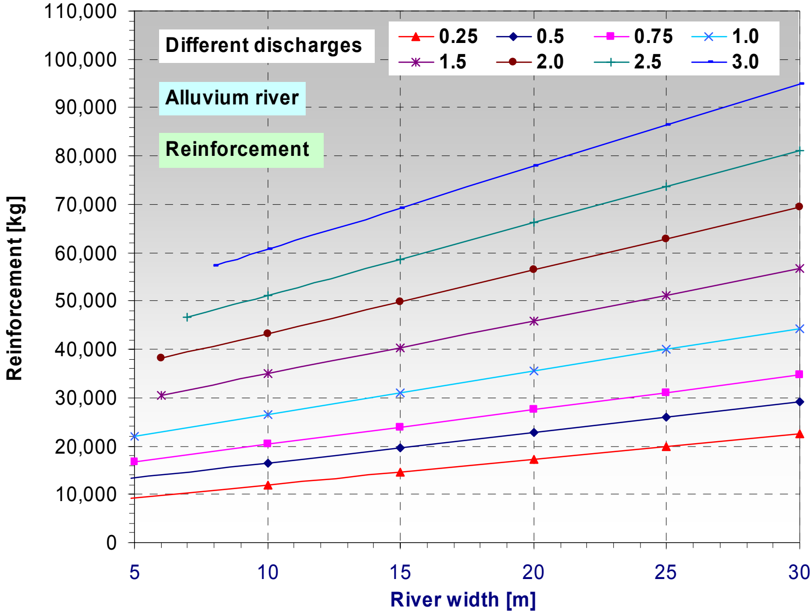
A2.9 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=45°



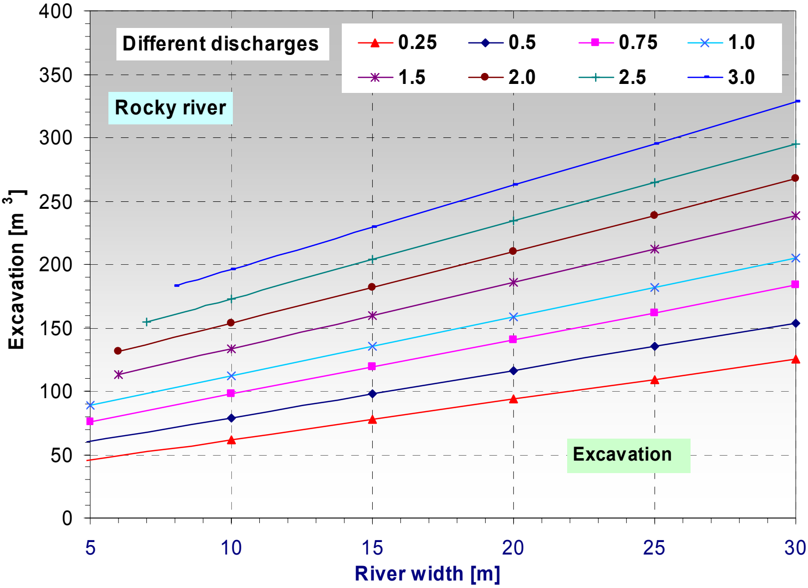


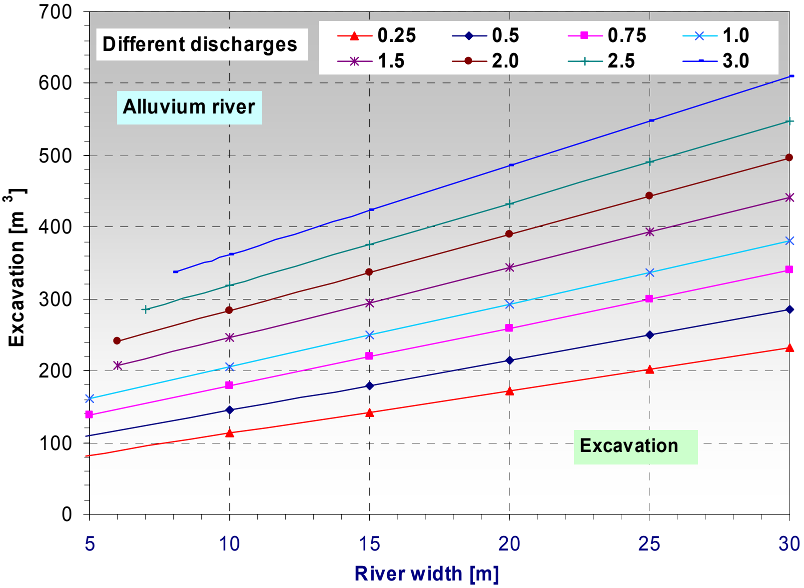
A2.9 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and $\beta=45^o$



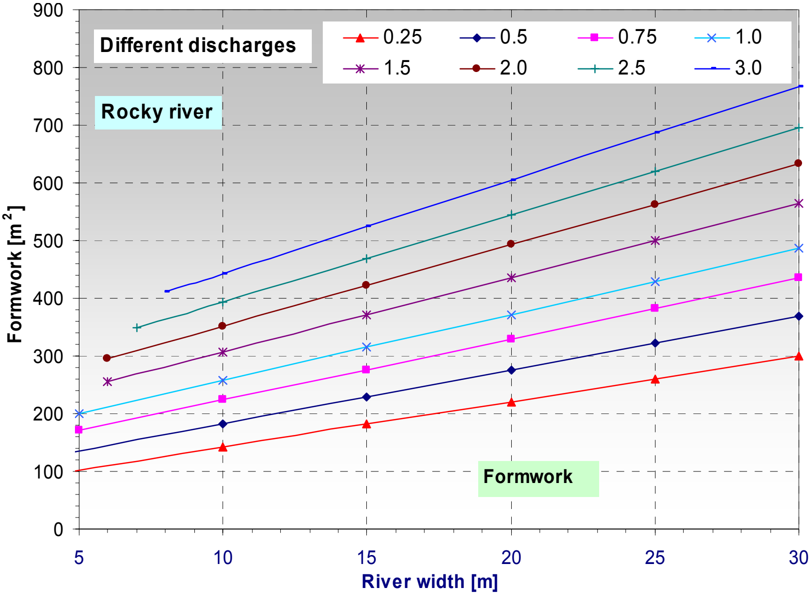


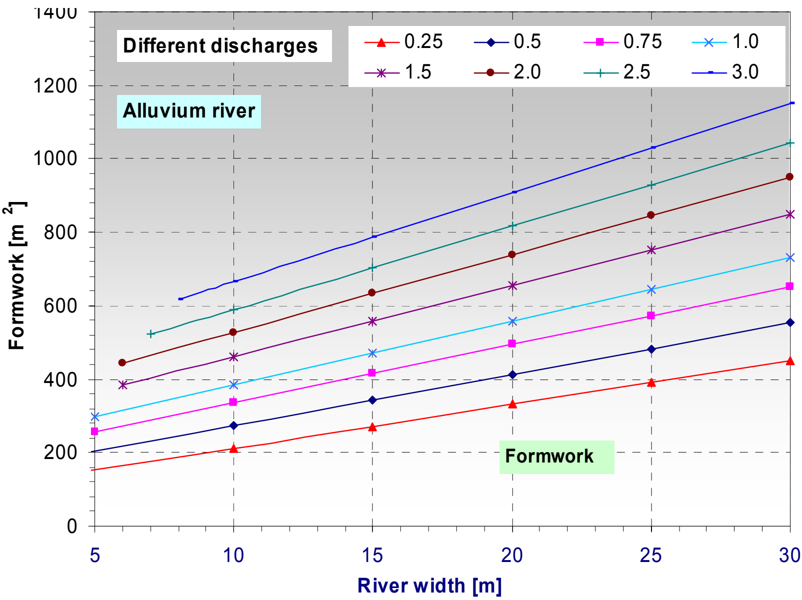
A2.9 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=45°



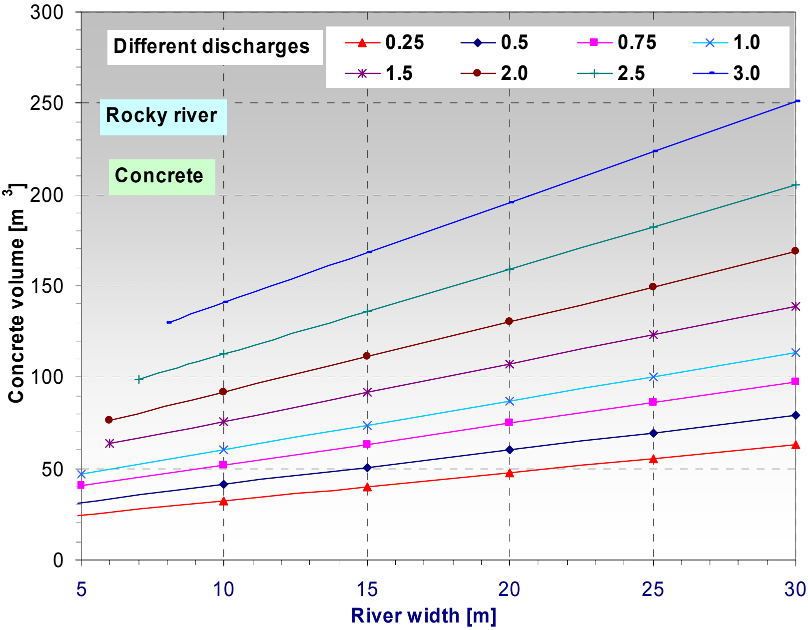


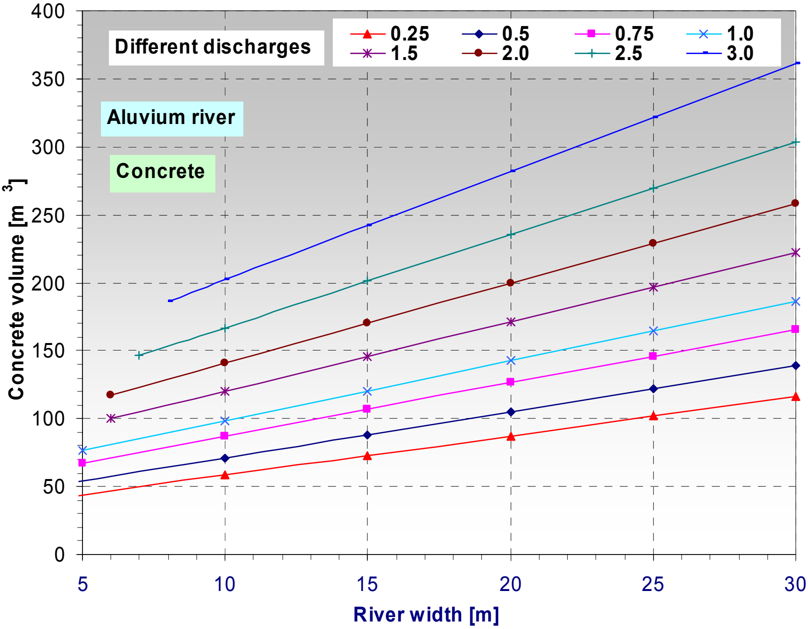
A2.9 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=1/2 and β=45°



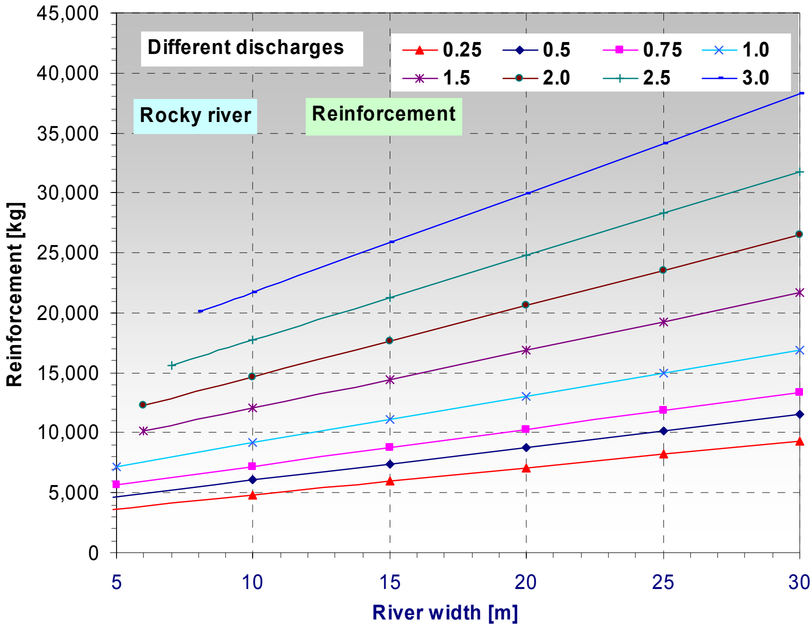


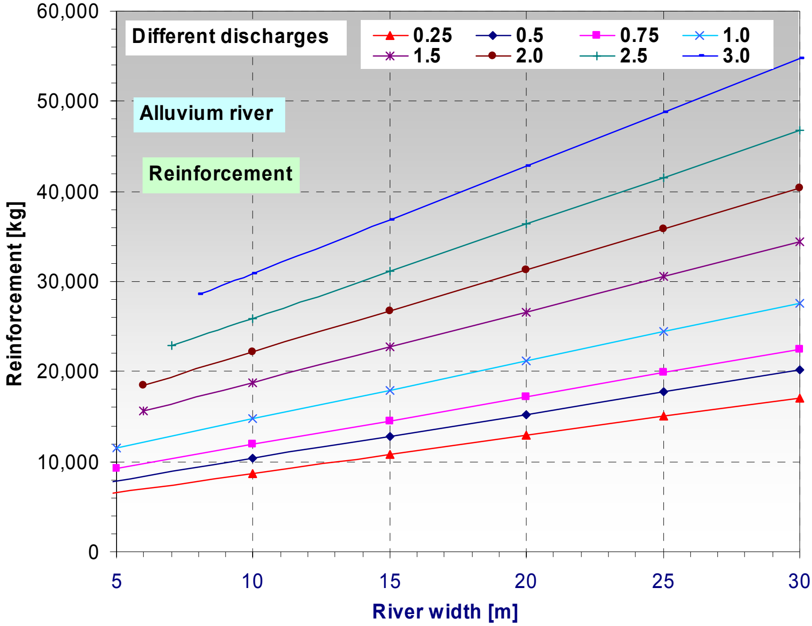
A2.10 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and β=30°



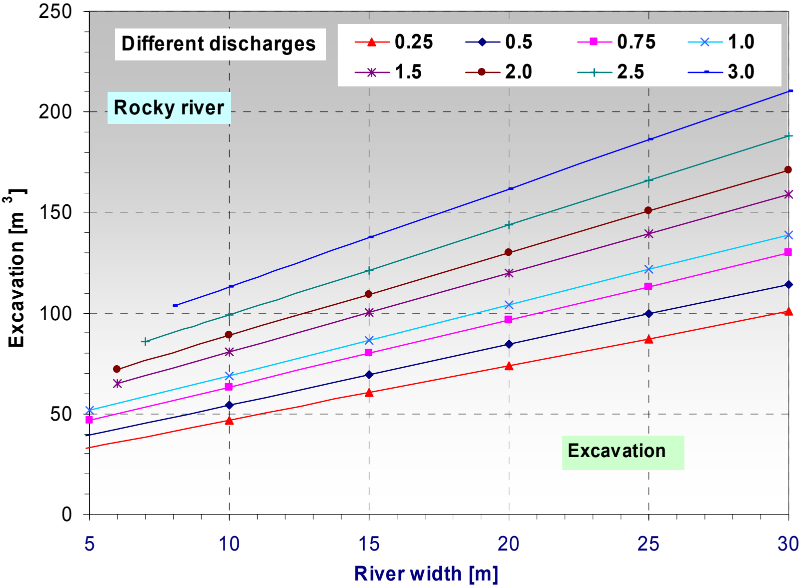


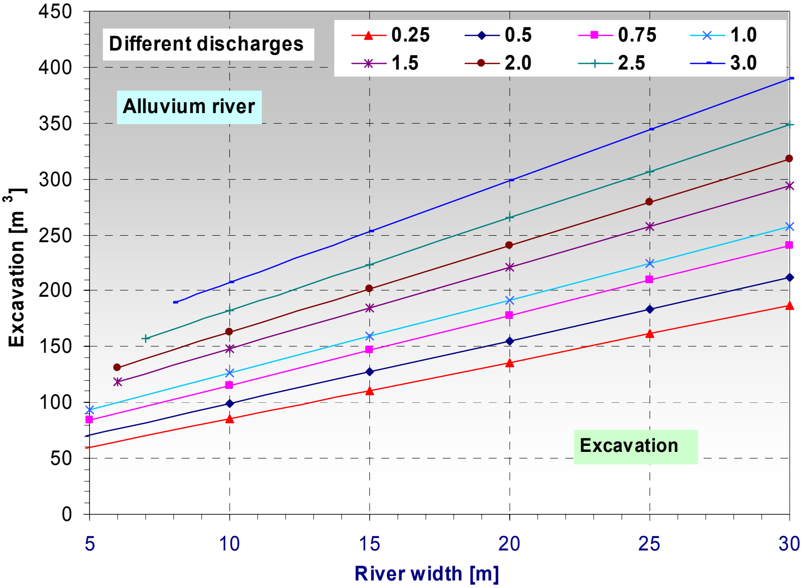
A2.10 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and β=30°



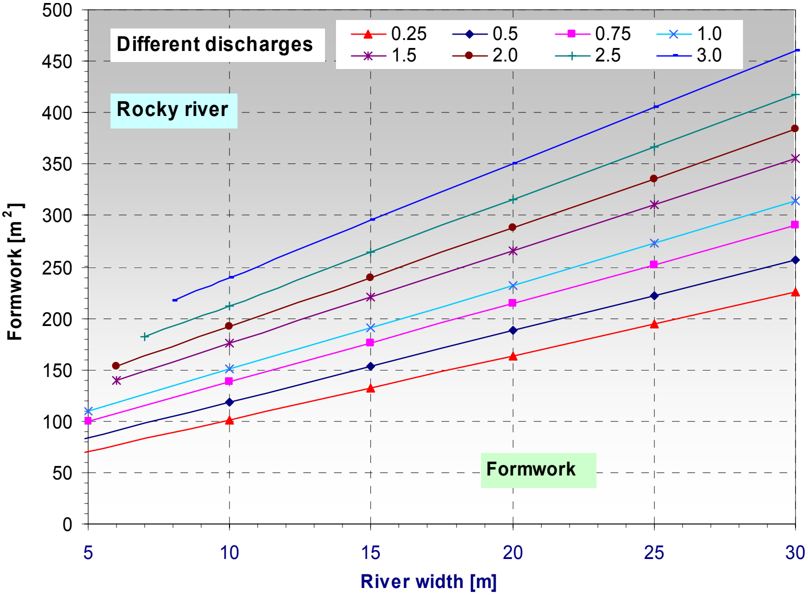


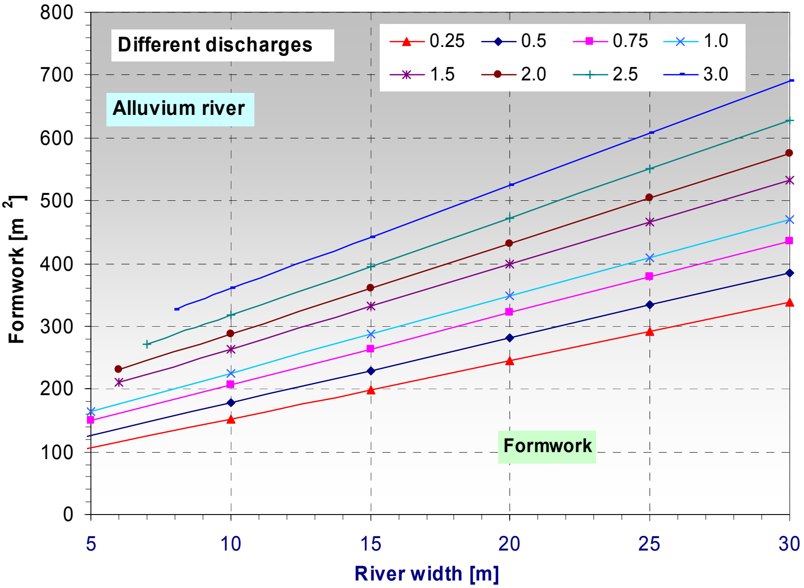
A2.10 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and $\beta=30^o$



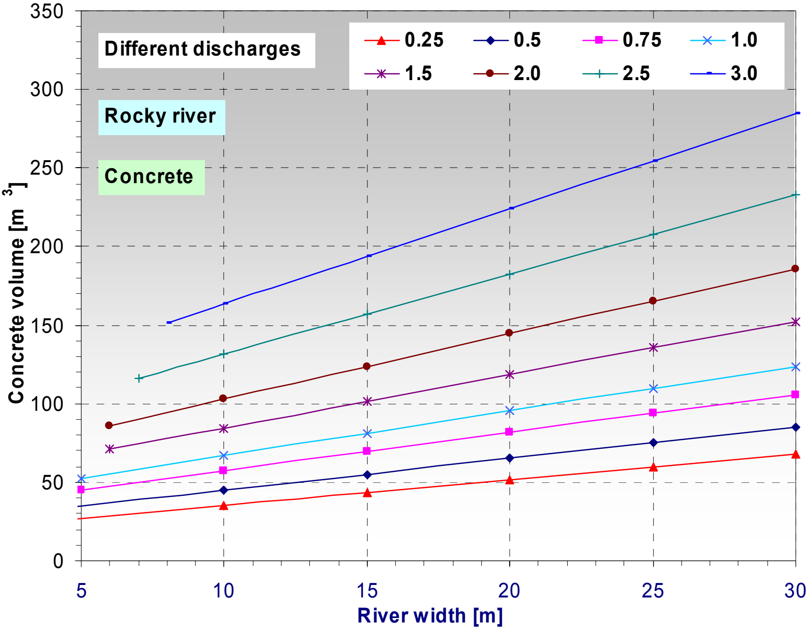


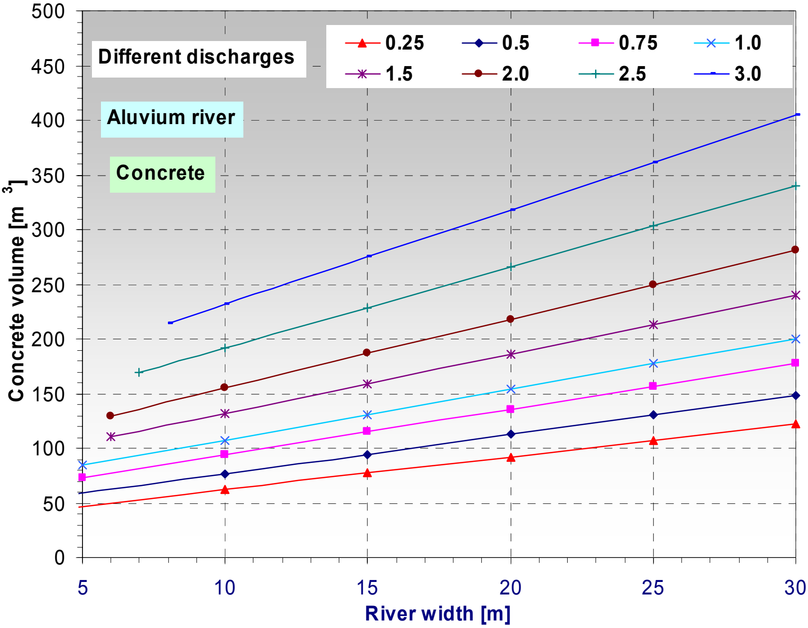
A2.10 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and β=30°



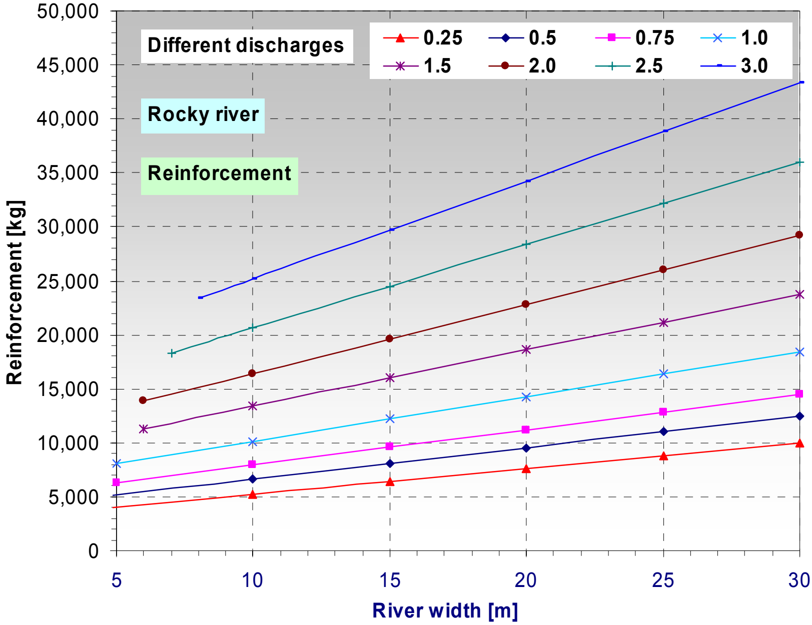


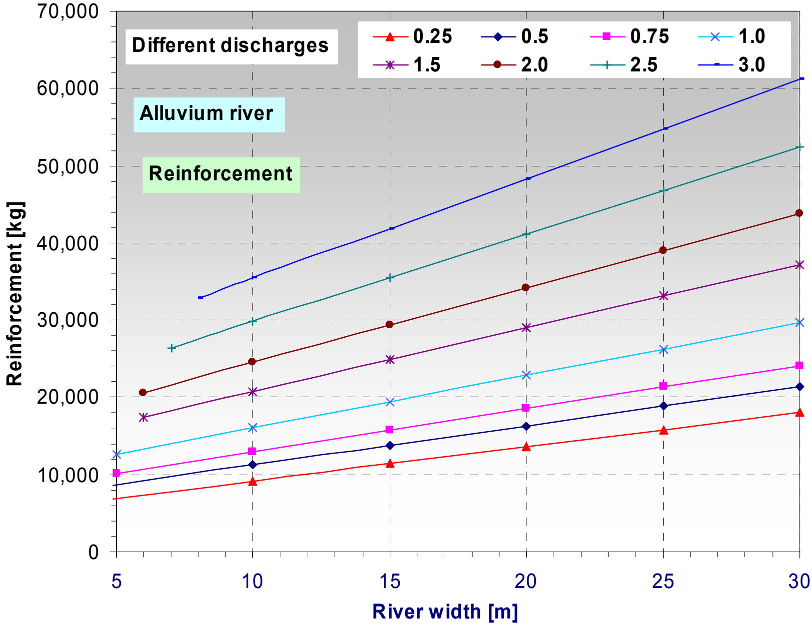
A2.11 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and β=35°



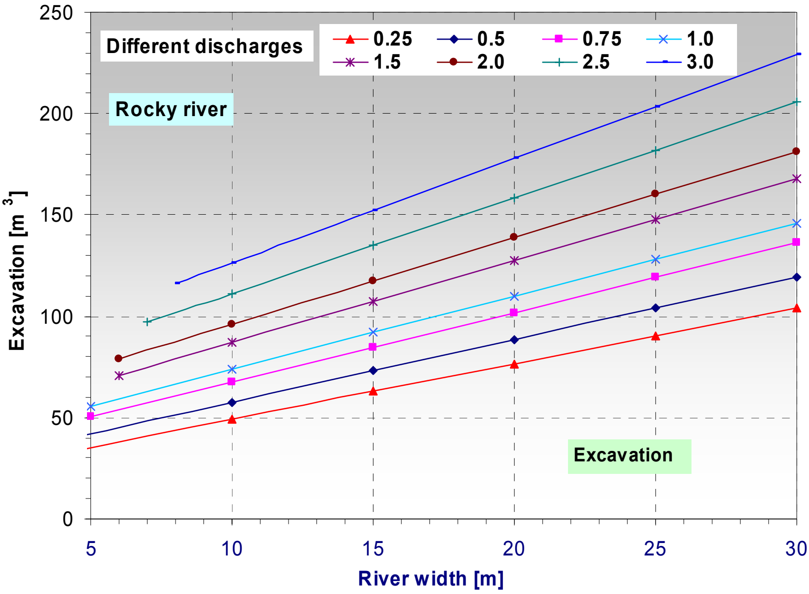


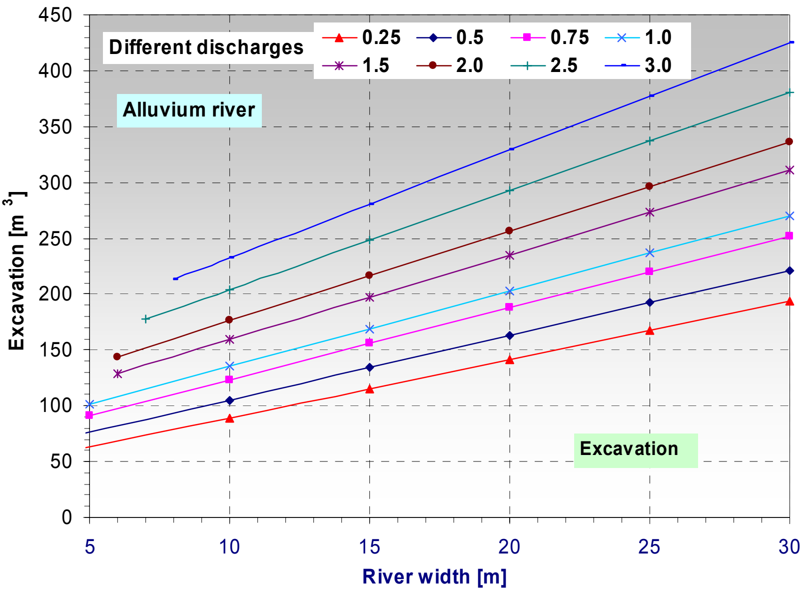
A2.11 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and $\beta=35^o$



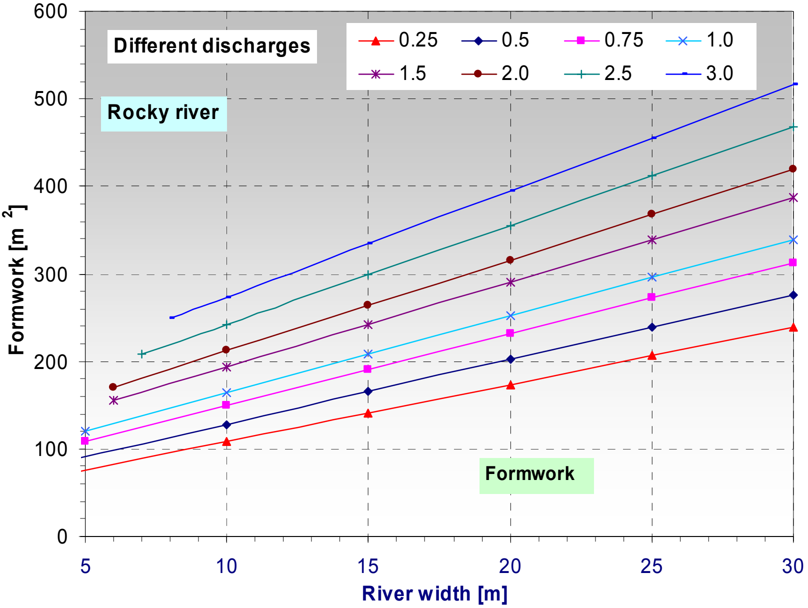


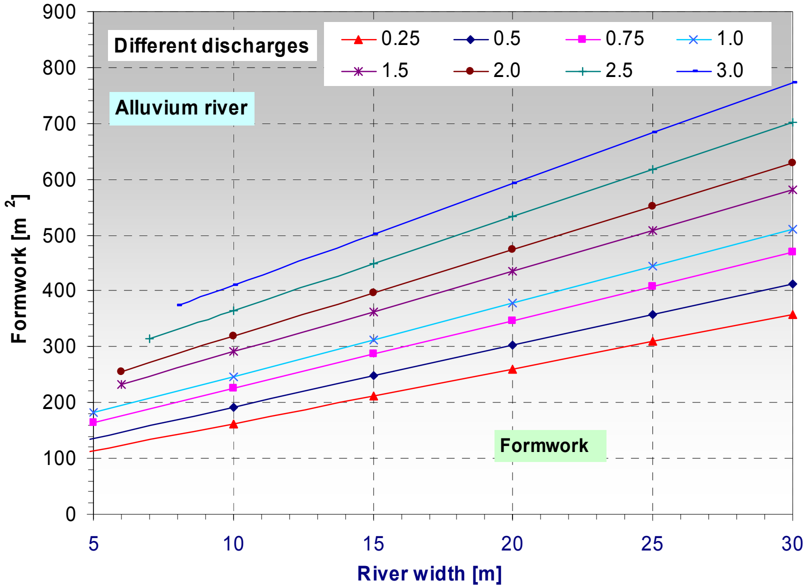
A2.11 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and β=35°



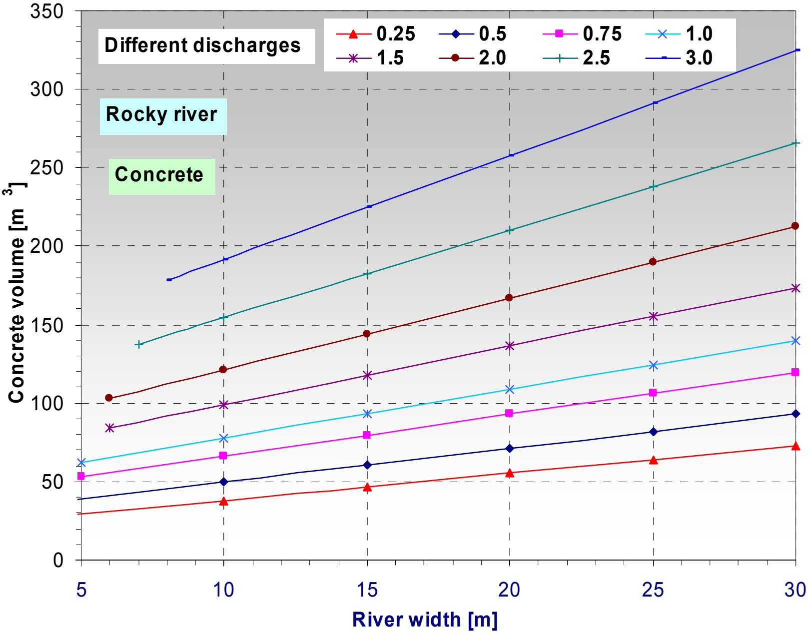


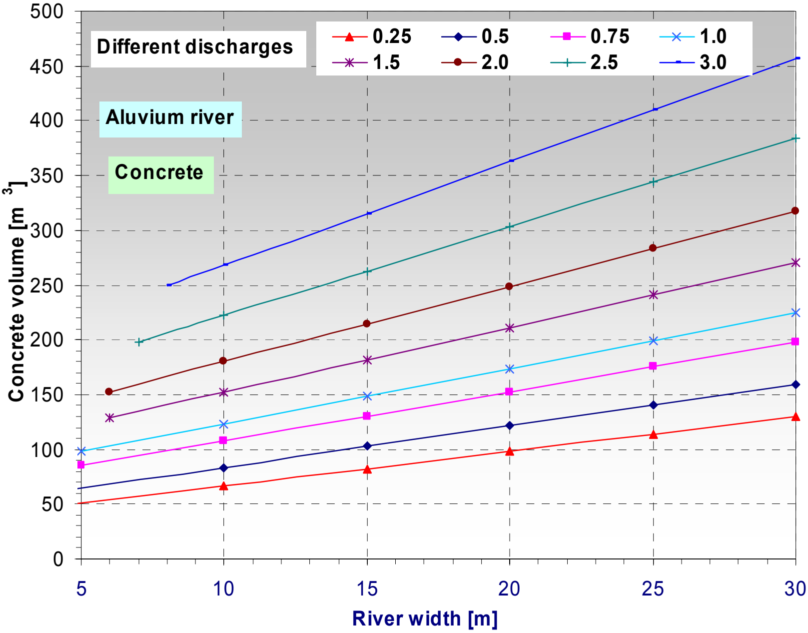
A2.11 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and β=35°



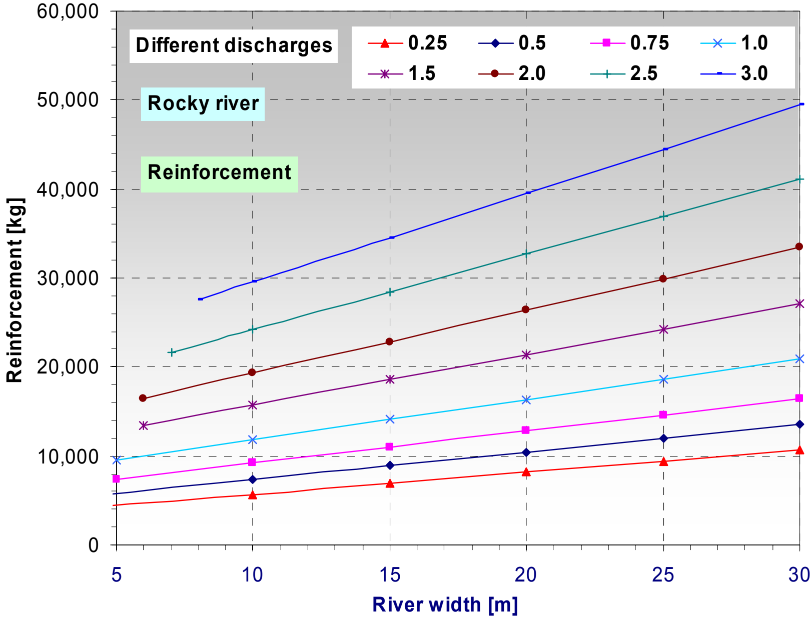


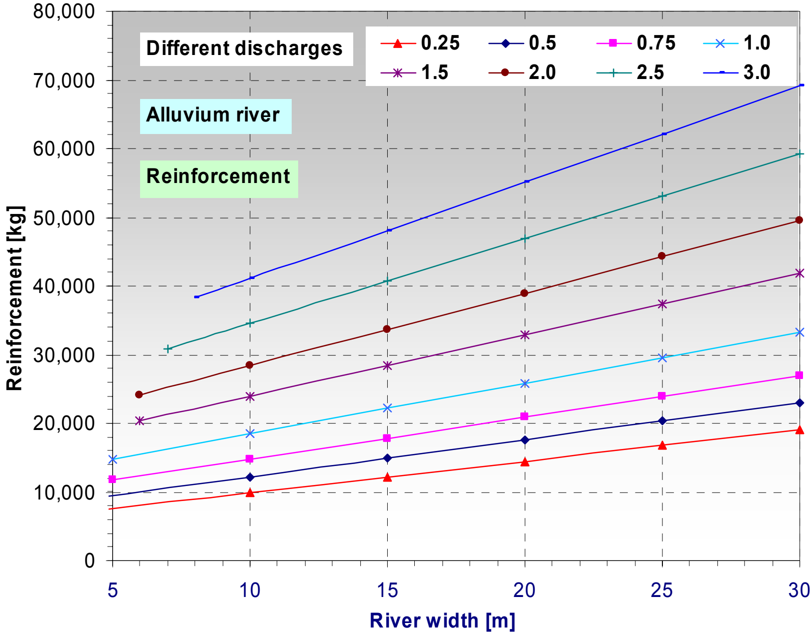
A2.12 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and β=40°



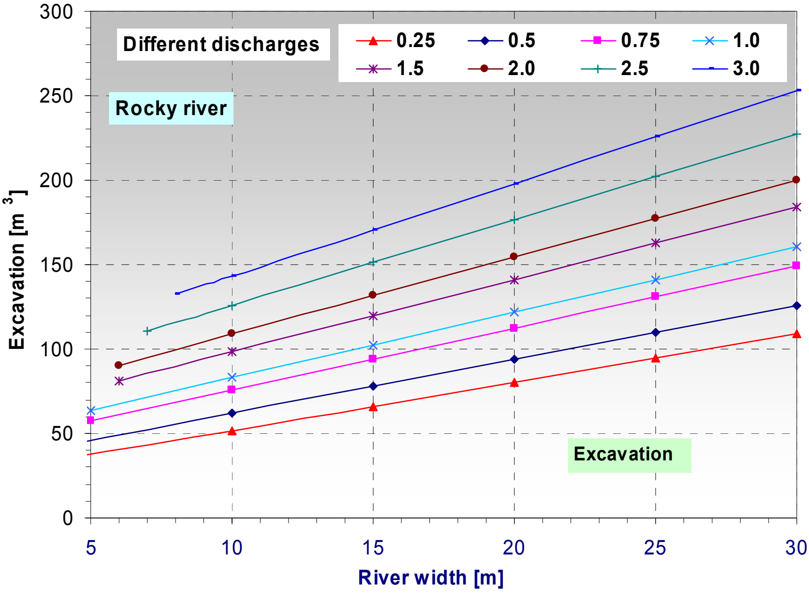


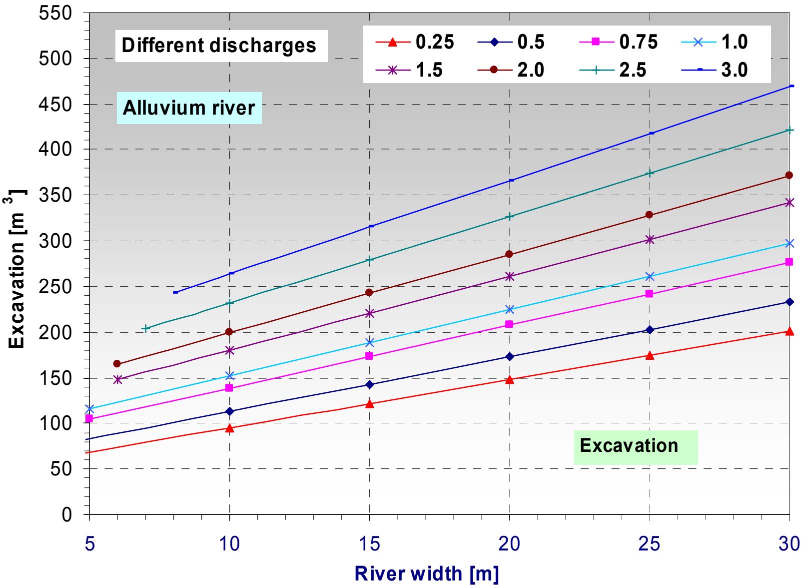
A2.12 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and β=40°



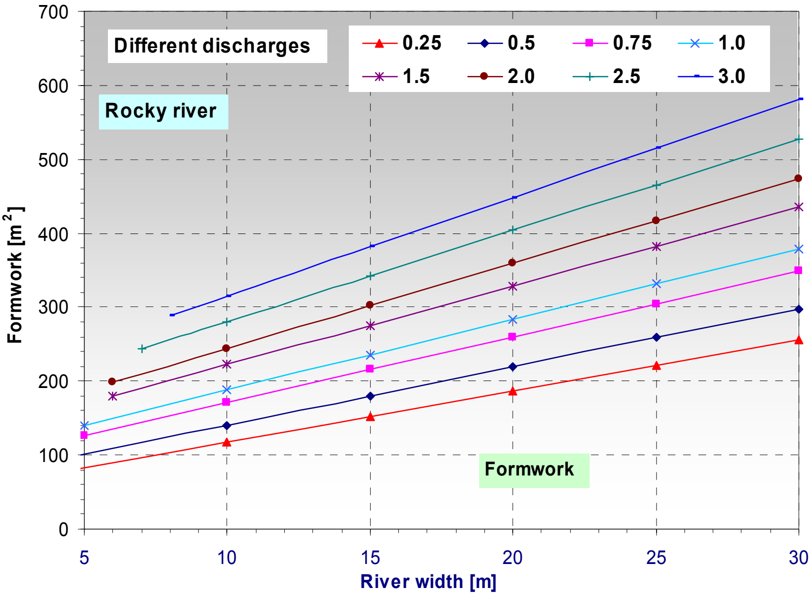


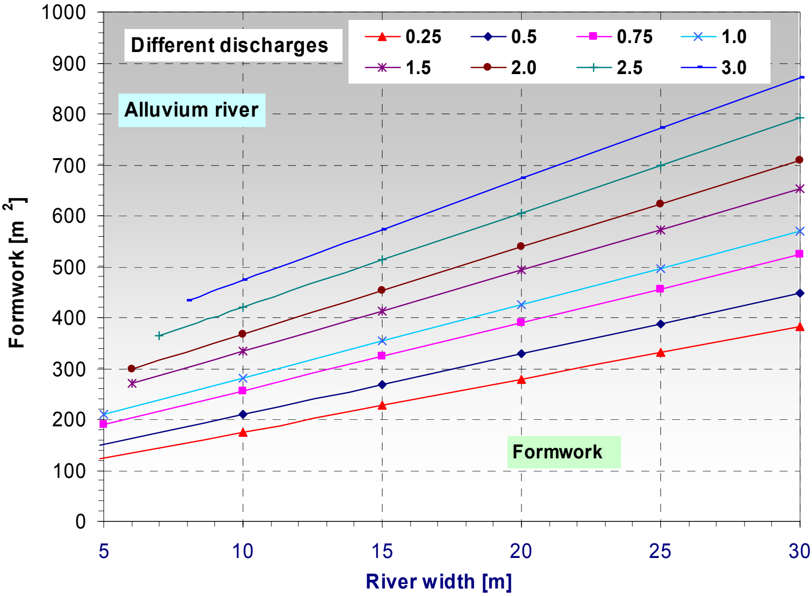
A2.12 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and β=40°



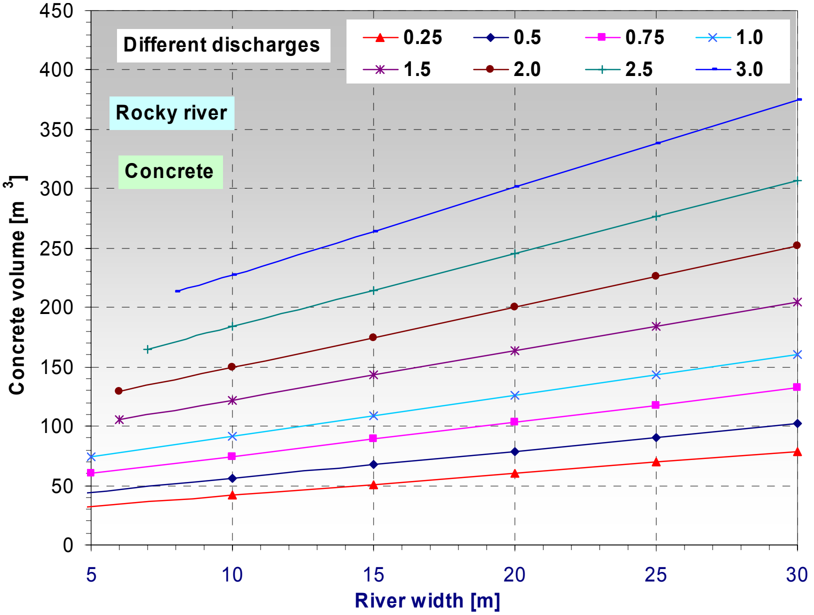


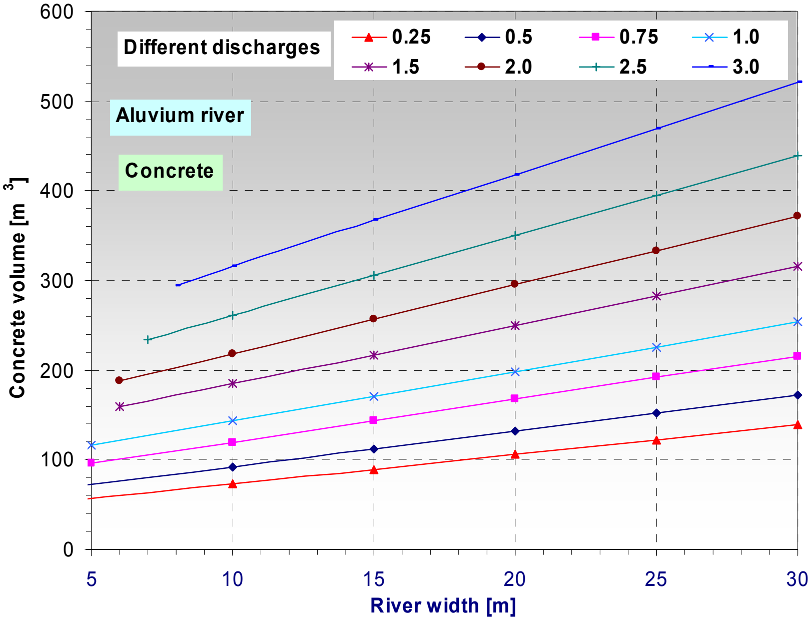
A2.12 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and β=40°



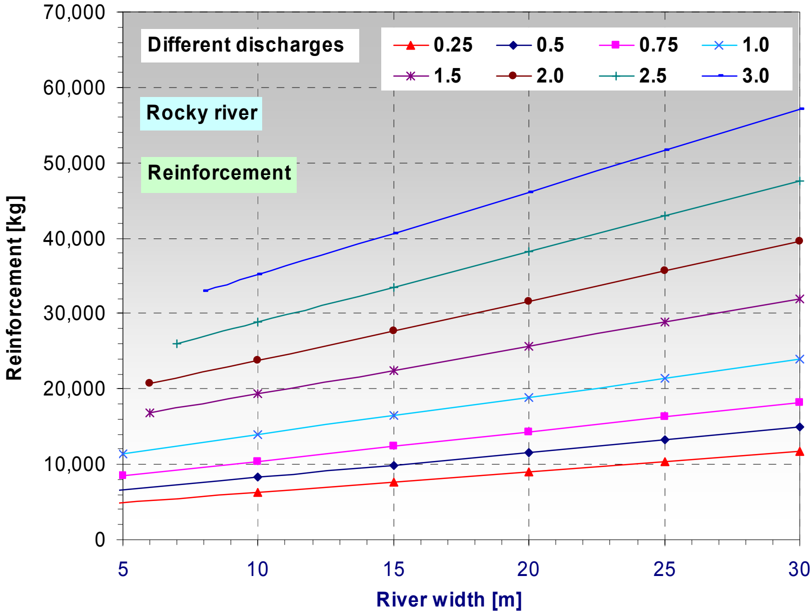


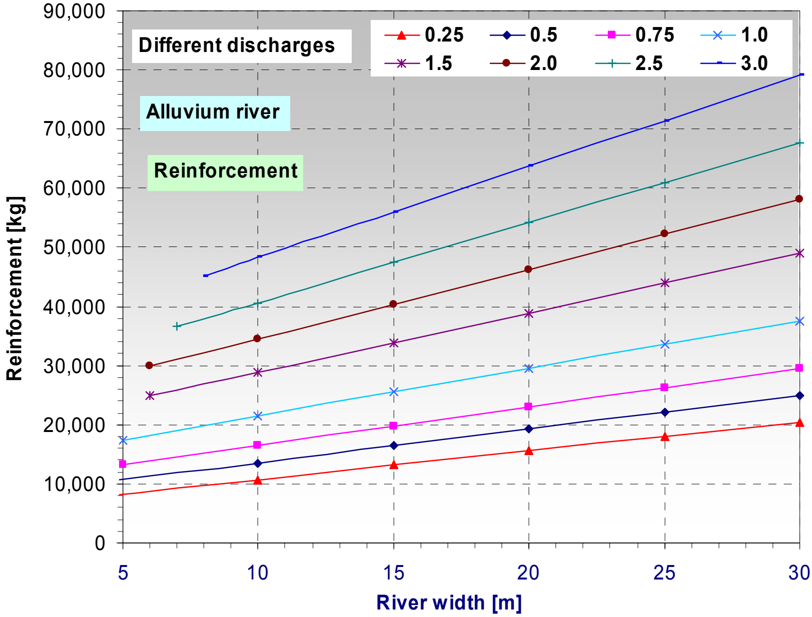
A2.13 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and β=45°



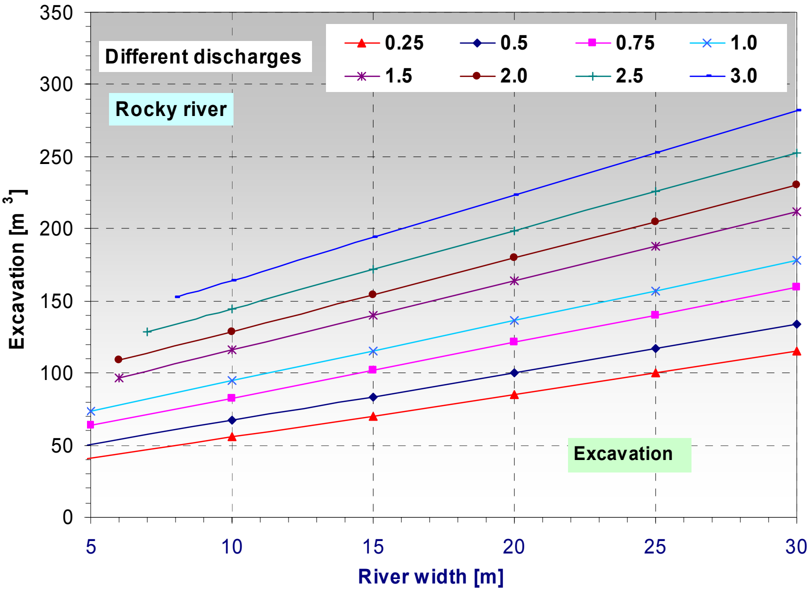


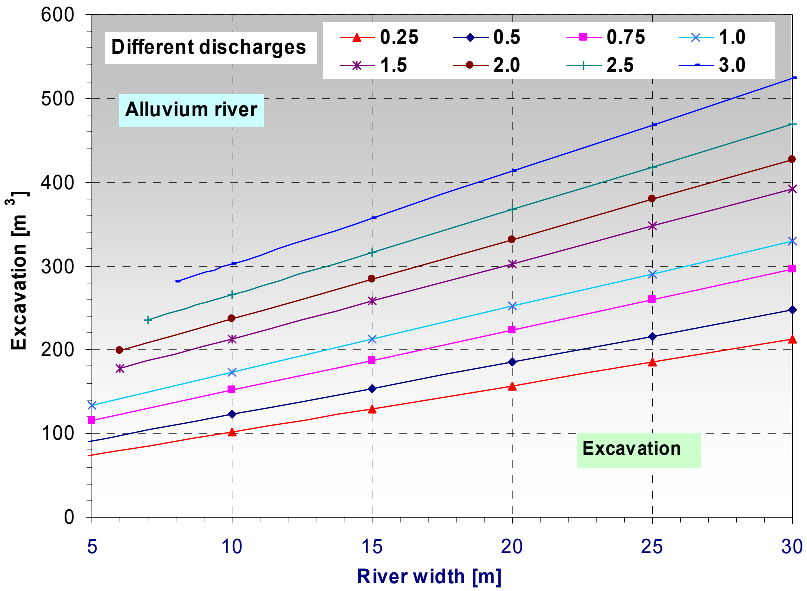
A2.13 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and $\beta=45^o$



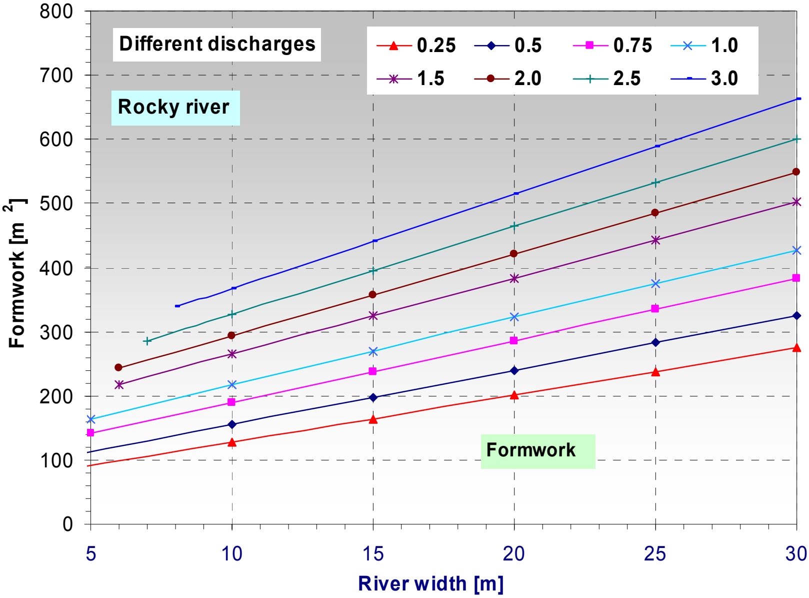


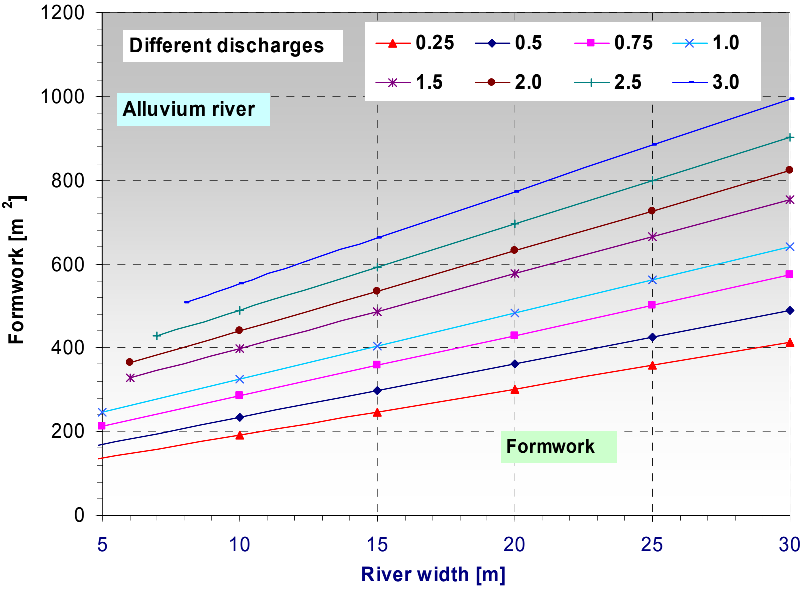
A2.13 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and $\beta=45^o$





A2.13 Concrete volume, reinforcement, excavation and formwork of Tyrolean intake as a function of river width and discharge (m^3/s) for a/b=2/3 and β=45°





Appendix B

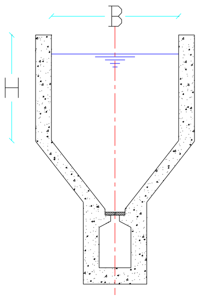
Standardization charts for settling basin (Sediment trap)

B1: Design tables of settling basin

B1.1: Width, length and height of settling basin as a function of discharge and design grain size as 0.2 and 0.3mm

Q: design discharge (m^3/s)
H: Height of settling basin (m)
B: Settlin basin width (m)
L: Settlin basin length (m)

d: design grain size (mm)



Values:
Width [B]
Height [H]
Length [L]

$Values = a * Q^b$

Settling basin, d=0.3mm

Values	a	b
B	2.6609	0.4584
H	2.0871	0.4583
L	23.4160	0.4584

Settling basin, d=0.2mm

Values	a	b
B	2.9043	0.4565
H	2.2776	0.4565
L	35.3650	0.3111

B1.2: Concrete volume, reinforcement, excavation and formwork of Bieri settling basin as a function of discharge and design grain size as 0.2 and 0.3mm

$Values = a * Q^2 + b * Q + c$

Values: concrete volume, reinforcment and excavation

Rocky bed

Bieri basin, d=0.3mm, concrete volume [m^3]

Comp.	a	b	c
Con.	27.02	90.28	8.76

Bieri basin, d=0.2mm, concrete volume [m^3]

Comp.	a	b	c
Con.	32.28	143.47	23.26

Bieri basin, d=0.3mm, reinforcement [kg]

Comp.	a	b	c
Reinf.	4212	13289	1271

Bieri basin, d=0.2mm, formwork [m^2]

Comp.	a	b	c
Reinf.	5854	26046	2574

Bieri basin, d=0.3mm, excavation [m^3]

Comp.	a	b	c
Exc.	53.7	681.1	-63.2

Bieri basin, d=0.2mm, excavation [m^3]

Comp.	a	b	c
Exc.	86.9	961.0	87.2

$Values = a * Q + b$

Values: formwork and backfilling

Bieri basin, d=0.3mm, formwork [m^2]

Comp.	a	b
Fw	590.7	95.4

Bieri basin, d=0.2mm, formwork [m^2]

Comp.	a	b
Fw	795.2	305.7

Bieri basin, d=0.3mm, backfilling [m^3]

Comp.	a	b
Fill.	319.5	-6.4

Bieri basin, d=0.2mm, backfilling [m^3]

Comp.	a	b
Fill.	460.7	58.8

B1.3: Concrete volume, reinforcement, excavation and formwork of Büchi settling basin as a function of discharge and design grain size as 0.2 and 0.3mm

Values = a * Q^2 +b * Q +c

Values: concrete volume, reinforcment and excavation

Rocky bed

Büchi basin, d=0.3mm, concrete volume [m^3]

Comp.	a	b	c
Con.	26.87	61.48	6.41

Büchi basin, d=0.2mm, concrete volume [m^3]

Comp.	a	b	c
Con.	35.78	93.25	25.02

Büchi basin, d=0.3mm, reinforcement [kg]

Comp.	a	b	c
Reinf.	4127	9046	-866

Büchi basin, d=0.2mm, formwork [m^2]

Comp.	a	b	c
Reinf.	6239	17654	-1112

Büchi basin, d=0.3mm, excavation [m^3]

Comp.	a	b	c
Exc.	84.4	414.0	-5.6

Büchi basin, d=0.2mm, excavation [m^3]

Comp.	a	b	c
Exc.	110.9	674.2	67.6

Values = a * Q + b

Values: formwork

Büchi basin, d=0.3mm, formwork [m^2]

Comp.	a	b
Fw	505.8	26.3

Büchi basin, d=0.2mm, formwork [m^2]

Comp.	a	b
Fw	697.4	174.5

Büchi basin, d=0.3mm, backfilling [m^3]

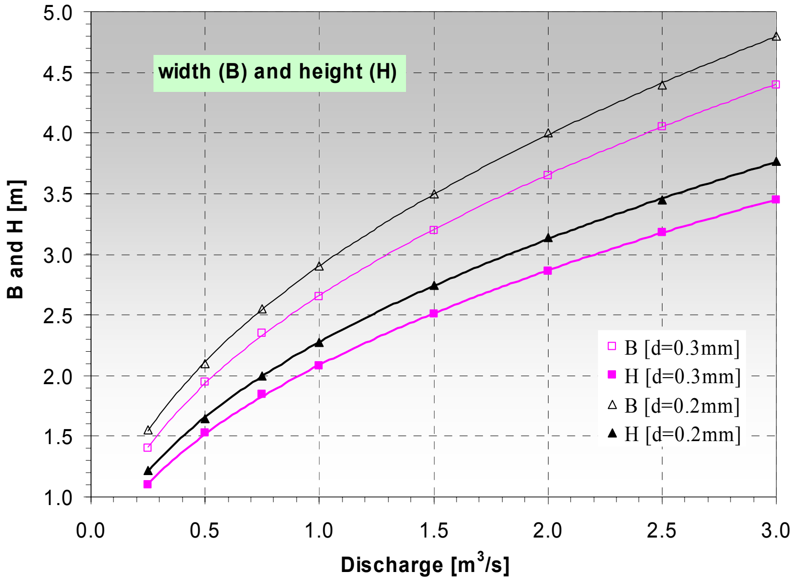
Comp.	a	b
Fill.	194.3	-12.2

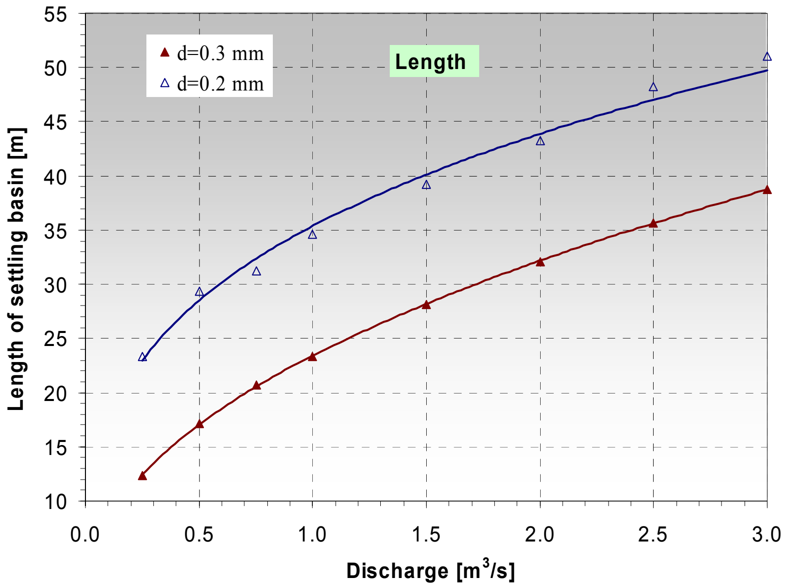
Büchi basin, d=0.2mm, backfilling [m^3]

Comp.	a	b
Fill.	283.9	24.0

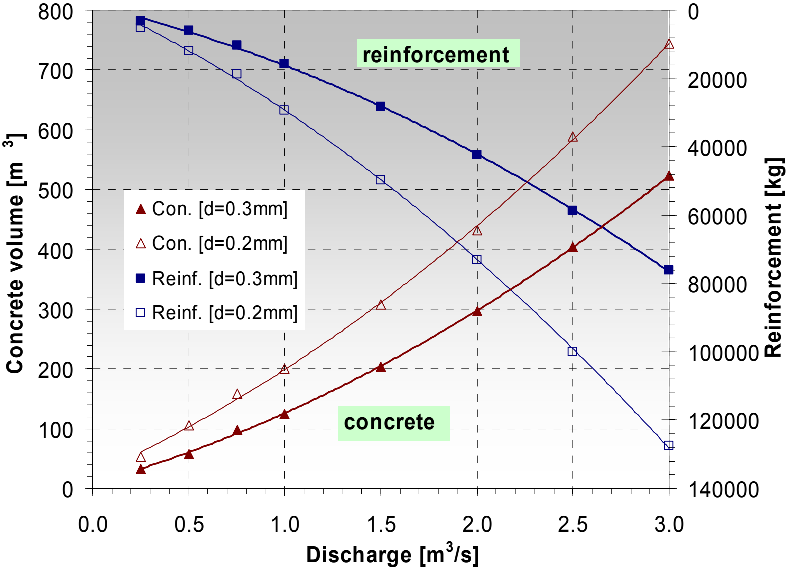
B2: Design charts of settling basin

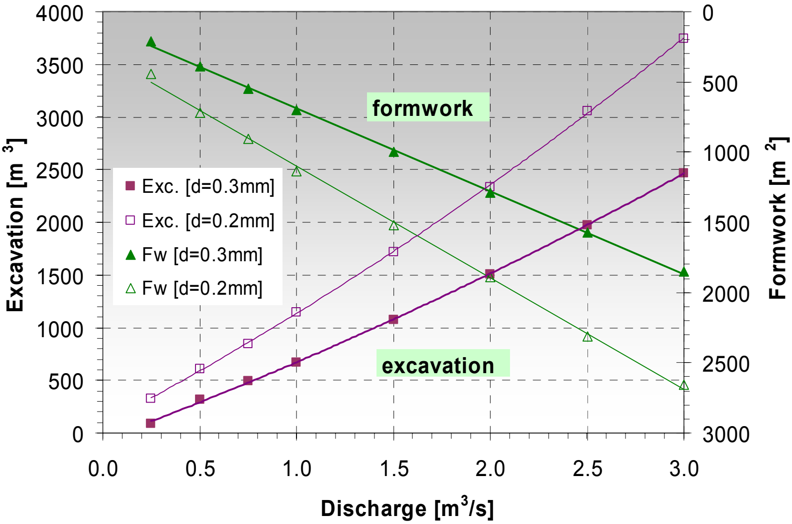
B2.1: Width, length and height of settling basin as a function of discharge and design grain size as 0.2 and 0.3mm



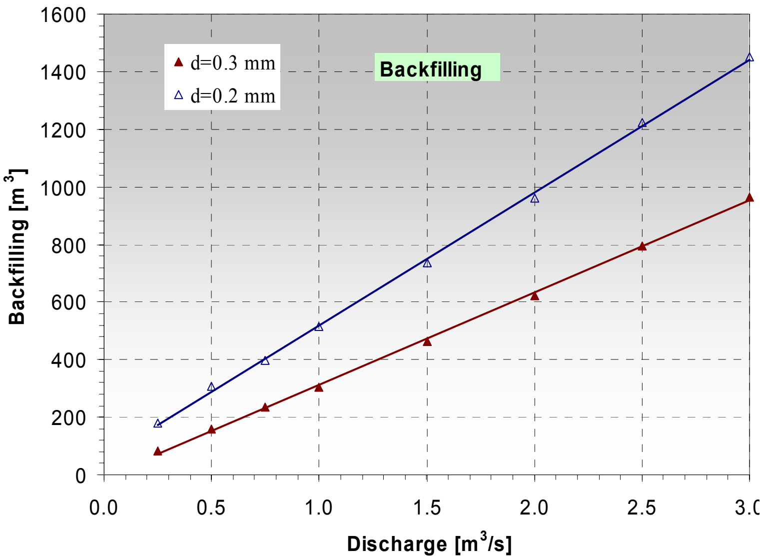


B2.2: Concrete volume, reinforcement, excavation, formwork and backfilling of Bieri settling basin as a function of discharge and design grain size as 0.2 and 0.3mm

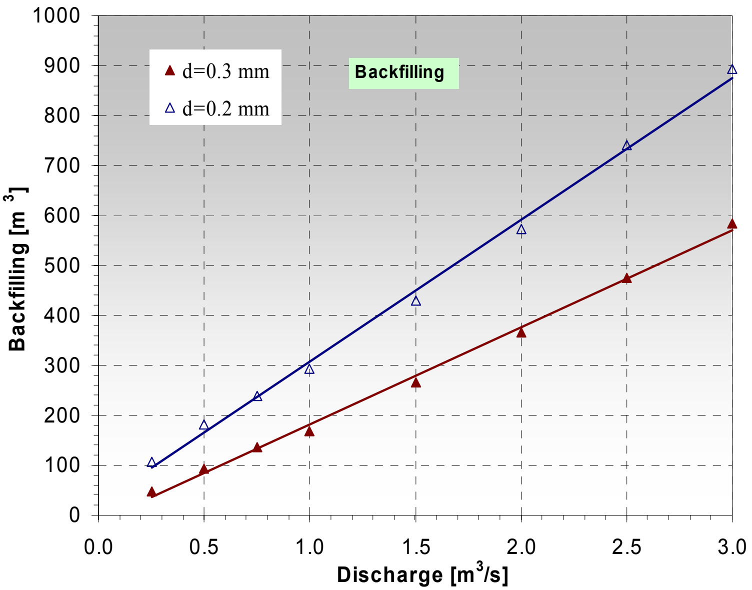




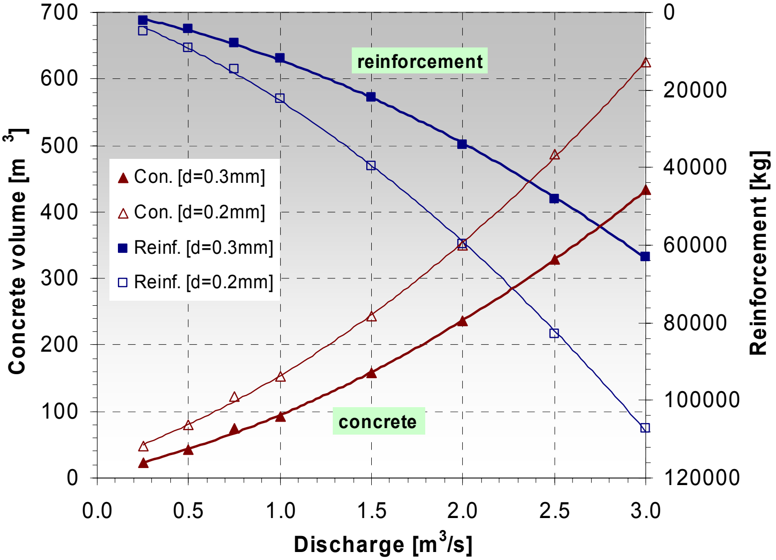
B2.2: Concrete volume, reinforcement, excavation, formwork and backfilling of Bieri settling basin as a function of discharge and design grain size as 0.2 and 0.3mm

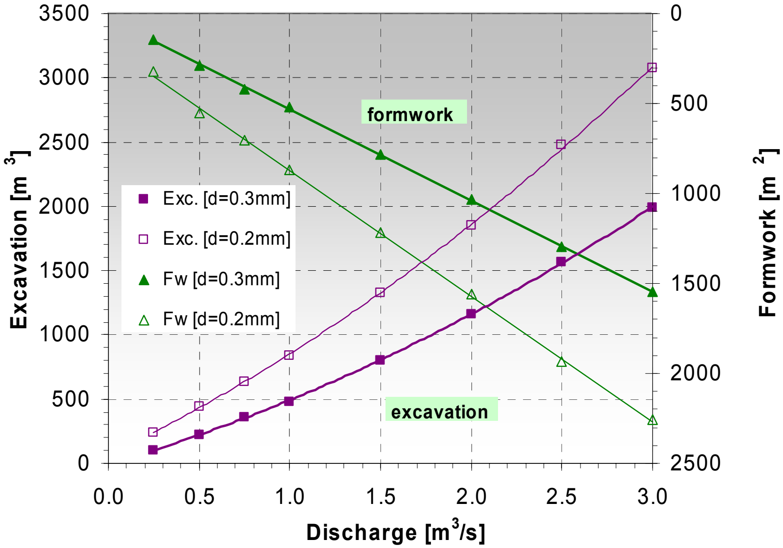


B2.3: Concrete volume, reinforcement, excavation, formwork and backfilling of Büchi settling basin as a function of discharge and design grain size as 0.2 and 0.3mm



B2.3: Concrete volume, reinforcement, excavation, formwork and backfilling of Büchi settling basin as a function of discharge and design grain size as 0.2 and 0.3mm





Appendix C

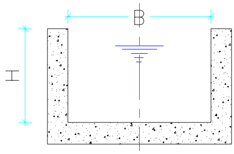
Standardization charts for headrace canal and pipe

C1: Design tables of headrace canal

C1.1: Open rectangular canal

C1.1.1: Width and height of open canal as a function of discharge and canal slope

Q: design discharge (m^3/s)
B: Open canal width (m)
H: Open canal height (m)
S: Open canal slope



Values = c * Q ^{d}

Values: width [B] , Height [H]

S=0.001

Values	c	d
B	1.3998	0.375
H	0.8234	0.375

S=0.002

Values	c	d
B	1.2292	0.375
H	0.7230	0.375

S=0.003

Values	c	d
B	1.1392	0.375
H	0.6701	0.375

S=0.004

Values	c	d
B	1.0794	0.375
H	0.6349	0.375

C1.1.2: Concrete volume, reinforcement, excavation, formwork and backfilling of open canal as a function of discharge and canal slope

Values = c * Q ^{d}

Rocky bed

Values: concrete volume, reinforcment, excavation, formwork and backfilling

S=0.001, concrete [m^3]

Comp.	c	d
Con.	0.9810	0.3195

S=0.002, concrete [m^3]

Comp.	c	d
Con.	0.8788	0.3131

S=0.001, reinf. [kg]

Comp.	c	d
Reinf.	94.8750	0.3195

S=0.002, reinf. [kg]

Comp.	c	d
Reinf.	84.9890	0.3131

S=0.001, exc. [m^3]

Comp.	c	d
Exc.	7.8390	0.3873

S=0.002, exc. [m^3]

Comp.	c	d
Exc.	6.8663	0.3637

S=0.001, formwork [m^2]

Comp.	c	d
Fw	4.1960	0.3230

S=0.002, formwork [m^2]

Comp.	c	d
Fw	3.7540	0.3170

S=0.001, backfill. [m^3]

Comp.	c	d
Bf	5.4873	0.3873

S=0.002, backfill. [m^3]

Comp.	c	d
Bf	4.8064	0.3637

For unit length of open canal

C1.1.2: Concrete volume, reinforcement, excavation, formwork and backfilling of open canal as a function of discharge and canal slope

Values = c * Q ^{d}

Rocky bed

Values: concrete volume, reinforcment, excavation, formwork and backfilling

S=0.003, concrete [m^3]

Comp.	c	d
Con.	0.8248	0.3091

S=0.004, concrete [m^3]

Comp.	c	d
Con.	0.7890	0.3061

S=0.002, reinf. [kg]

Comp.	c	d
Reinf.	79.7730	0.3091

S=0.004, reinf. [kg]

Comp.	c	d
Reinf.	76.3050	0.3061

S=0.003, exc. [m^3]

Comp.	c	d
Exc.	6.3782	0.3500

S=0.004, exc. [m^3]

Comp.	c	d
Exc.	6.0632	0.3403

S=0.003, formwork [m^2]

Comp.	c	d
Fw	3.5208	0.3132

S=0.004, formwork [m^2]

Comp.	c	d
Fw	3.3658	0.3104

S=0.002, backfill. [m^3]

Comp.	c	d
Bf	4.4647	0.3500

S=0.004, backfill. [m^3]

Comp.	c	d
Bf	4.2442	0.3403

For unit length of open canal

C1.1.3: Hydraulic calculations of open and buried canal

Q (m^3/s)	V (m/s)	n	A (m^2)	Y (m)	B (m)	P (m)	S	Fr	H (m)	FB (m)
0.25	0.72	0.0154	0.35	0.42	0.83	1.66	0.001	0.36	0.49	0.07
0.50	0.86	0.0154	0.58	0.54	1.08	2.16	0.001	0.37	0.63	0.10
0.75	0.95	0.0154	0.79	0.63	1.26	2.51	0.001	0.38	0.74	0.11
1.00	1.02	0.0154	0.98	0.70	1.40	2.80	0.001	0.39	0.82	0.12
1.50	1.13	0.0154	1.33	0.81	1.63	3.26	0.001	0.40	0.96	0.14
2.00	1.21	0.0154	1.65	0.91	1.82	3.63	0.001	0.41	1.07	0.16
2.50	1.28	0.0154	1.95	0.99	1.97	3.95	0.001	0.41	1.16	0.17
3.00	1.34	0.0154	2.23	1.06	2.11	4.23	0.001	0.42	1.24	0.19

Q (m^3/s)	V (m/s)	n	A (m^2)	Y (m)	B (m)	P (m)	S	Fr	H (m)	FB (m)
0.25	0.94	0.0154	0.27	0.37	0.73	1.46	0.002	0.49	0.43	0.06
0.50	1.11	0.0154	0.45	0.47	0.95	1.90	0.002	0.52	0.56	0.08
0.75	1.23	0.0154	0.61	0.55	1.10	2.21	0.002	0.53	0.65	0.10
1.00	1.32	0.0154	0.76	0.61	1.23	2.46	0.002	0.54	0.72	0.11
1.50	1.46	0.0154	1.02	0.72	1.43	2.86	0.002	0.55	0.84	0.13
2.00	1.57	0.0154	1.27	0.80	1.59	3.19	0.002	0.56	0.94	0.14
2.50	1.66	0.0154	1.50	0.87	1.73	3.47	0.002	0.57	1.02	0.15
3.00	1.74	0.0154	1.72	0.93	1.86	3.71	0.002	0.58	1.09	0.16

Q (m^3/s)	V (m/s)	n	A (m^2)	Y (m)	B (m)	P (m)	S	Fr	H (m)	FB (m)
0.25	1.09	0.0154	0.23	0.34	0.68	1.35	0.003	0.60	0.40	0.06
0.50	1.30	0.0154	0.39	0.44	0.88	1.76	0.003	0.62	0.52	0.08
0.75	1.43	0.0154	0.52	0.51	1.02	2.05	0.003	0.64	0.60	0.09
1.00	1.54	0.0154	0.65	0.57	1.14	2.28	0.003	0.65	0.67	0.10
1.50	1.71	0.0154	0.88	0.66	1.33	2.65	0.003	0.67	0.78	0.12
2.00	1.83	0.0154	1.09	0.74	1.48	2.95	0.003	0.68	0.87	0.13
2.50	1.94	0.0154	1.29	0.80	1.61	3.21	0.003	0.69	0.94	0.14
3.00	2.03	0.0154	1.48	0.86	1.72	3.44	0.003	0.70	1.01	0.15

Q (m^3/s)	V (m/s)	n	A (m^2)	Y (m)	B (m)	P (m)	S	Fr	H (m)	FB (m)
0.25	1.21	0.0154	0.21	0.32	0.64	1.28	0.004	0.68	0.38	0.06
0.50	1.44	0.0154	0.35	0.42	0.83	1.66	0.004	0.71	0.49	0.07
0.75	1.60	0.0154	0.47	0.48	0.97	1.94	0.004	0.73	0.57	0.09
1.00	1.72	0.0154	0.58	0.54	1.08	2.16	0.004	0.75	0.63	0.10
1.50	1.90	0.0154	0.79	0.63	1.26	2.51	0.004	0.77	0.74	0.11
2.00	2.04	0.0154	0.98	0.70	1.40	2.80	0.004	0.78	0.82	0.12
2.50	2.16	0.0154	1.16	0.76	1.52	3.04	0.004	0.79	0.90	0.13
3.00	2.26	0.0154	1.33	0.81	1.63	3.26	0.004	0.80	0.96	0.14

Q: design discharge (m^3/s)
V: velocity (m/s)
n: Manning coefficient
A: flow area (m^2)
Y: water depth (m)
FB: freeboard (m)

B: width (m)
P: wet perimeter (m)
S: slope
Fr: Froude number
H: height of structure (m)

C1.2: Buried rectangular canal

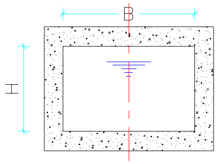
C1.2.1: Width and height of buried canal as a function of discharge and canal slope

Q: design discharge (m^3/s)

B: Buried canal width (m)

H: Buried canal height (m)

S: Buried canal slope



Values = c * Q ^{d}

Values: width [B] , Height [H]

S=0.001

Values	c	d
B	1.3998	0.375
H	0.8234	0.375

S=0.002

Values	c	d
B	1.2292	0.375
H	0.7230	0.375

S=0.003

Values	c	d
B	1.1392	0.375
H	0.6701	0.375

S=0.004

Values	c	d
B	1.0794	0.375
H	0.6349	0.375

C1.2.2: Concrete volume, reinforcement, excavation, formwork and backfilling of buried canal as a function of discharge and canal slope

Values = c * Q ^{d}

Rocky bed

Values: concrete volume, reinforcment, excavation, formwork and backfilling

S=0.001, concrete [m^3]

Comp.	c	d
Con.	1.5085	0.3030

S=0.002, concrete [m^3]

Comp.	c	d
Con.	1.3592	0.2952

S=0.001, reinf. [kg]

Comp.	c	d
Reinf.	145.89	0.3030

S=0.002, reinf. [kg]

Comp.	c	d
Reinf.	131.45	0.2952

S=0.001, exc. [m^3]

Comp.	c	d
Exc.	9.2694	0.3530

S=0.002, exc. [m^3]

Comp.	c	d
Exc.	8.2157	0.3292

S=0.001, formwork [m^2]

Comp.	c	d
Fw	7.8481	0.3195

S=0.002, formwork [m^2]

Comp.	c	d
Fw	7.0302	0.3131

S=0.001, backfill. [m^3]

Comp.	c	d
Bf	6.4886	0.3530

S=0.002, backfill. [m^3]

Comp.	c	d
Bf	5.7510	0.3292

For unit length of buried canal

C1.2.2: Concrete volume, reinforcement, excavation, formwork and backfilling of buried canal as a function of discharge and canal slope

Values = c * Q^{d}

Rocky bed

Values: concrete volume, reinforcment, excavation, formwork and backfilling

S=0.003, concrete [m^3]

Comp.	c	d
Con.	1.2804	0.2904

S=0.004, concrete [m^3]

Comp.	c	d
Con.	1.2280	0.2869

S=0.002, reinf. [kg]

Comp.	c	d
Reinf.	123.83	0.2904

S=0.004, reinf. [kg]

Comp.	c	d
Reinf.	118.77	0.2869

S=0.003, exc. [m^3]

Comp.	c	d
Exc.	7.6847	0.3155

S=0.004, exc. [m^3]

Comp.	c	d
Exc.	7.3412	0.3058

S=0.003, formwork [m^2]

Comp.	c	d
Fw	6.5988	0.3091

S=0.004, formwork [m^2]

Comp.	c	d
Fw	6.3119	0.3061

S=0.002, backfill. [m^3]

Comp.	c	d
Bf	5.3793	0.3155

S=0.004, backfill. [m^3]

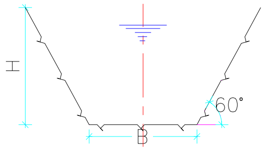
Comp.	c	d
Bf	5.1389	0.3058

For unit length of buried canal

C1.3: Open rock canal

C1.3.1: Width and height of open rock canal as a function of discharge and canal slope

Q: design discharge (m^3/s)
B: Rocky canal width (m)
H: Rocky canal height (m)
S: Rocky canal slope



Values = c * Q ^{d}

Values: width [B] , Height [H]

S=0.001

Values	c	d
B	1.0753	0.375
H	1.0963	0.375

S=0.002

Values	c	d
B	0.9442	0.375
H	0.9627	0.375

S=0.003

Values	c	d
B	0.8751	0.375
H	0.8922	0.375

S=0.004

Values	c	d
B	0.8292	0.375
H	0.8454	0.375

C1.3.2: Excavation and backfilling of open rock canal as a function of discharge and canal slope

Values = c * Q ^{d}

Rocky bed

Values: excavation and backfilling

S=0.001, excavation [m^3]

Comp.	c	d
Exc.	8.5830	0.5279

S=0.002, excavation [m^3]

Comp.	c	d
Exc.	7.7662	0.5192

S=0.001, backfilling [m^3]

Comp.	c	d
Bf	6.0081	0.5279

S=0.002, backfilling [m^3]

Comp.	c	d
Bf	5.4363	0.5192

S=0.003, excavation [m^3]

Comp.	c	d
Exc.	6.4617	0.4957

S=0.004, excavation [m^3]

Comp.	c	d
Exc.	6.0163	0.4870

S=0.002, backfilling [m^3]

Comp.	c	d
Bf	4.5232	0.4957

S=0.004, backfilling [m^3]

Comp.	c	d
Bf	4.2114	0.4870

For unit length of open rock canal

C1.3.3: Hydraulic calculations of open rock canal

Q (m^3/s)	V (m/s)	n	α (°)	A (m^2)	Y (m)	B (m)	S	Fr	H (m)	FB (m)
0.25	0.47	0.029	60	0.53	0.55	0.64	0.001	0.23	0.65	0.10
0.50	0.56	0.029	60	0.89	0.72	0.83	0.001	0.24	0.85	0.13
0.75	0.62	0.029	60	1.21	0.84	0.97	0.001	0.25	0.98	0.15
1.00	0.67	0.029	60	1.50	0.93	1.08	0.001	0.25	1.10	0.16
1.50	0.74	0.029	60	2.04	1.08	1.25	0.001	0.26	1.28	0.19
2.00	0.79	0.029	60	2.53	1.21	1.39	0.001	0.27	1.42	0.21
2.50	0.84	0.029	60	2.99	1.31	1.52	0.001	0.27	1.55	0.23
3.00	0.88	0.029	60	3.43	1.41	1.62	0.001	0.27	1.66	0.25

Q (m^3/s)	V (m/s)	n	α (°)	A (m^2)	Y (m)	B (m)	S	Fr	H (m)	FB (m)
0.25	0.61	0.029	60	0.41	0.49	0.56	0.002	0.32	0.57	0.09
0.50	0.73	0.029	60	0.69	0.63	0.73	0.002	0.34	0.74	0.11
0.75	0.80	0.029	60	0.93	0.73	0.85	0.002	0.35	0.86	0.13
1.00	0.86	0.029	60	1.16	0.82	0.94	0.002	0.35	0.96	0.14
1.50	0.95	0.029	60	1.57	0.95	1.10	0.002	0.36	1.12	0.17
2.00	1.03	0.029	60	1.95	1.06	1.22	0.002	0.37	1.25	0.19
2.50	1.08	0.029	60	2.30	1.15	1.33	0.002	0.37	1.36	0.20
3.00	1.14	0.029	60	2.64	1.24	1.43	0.002	0.38	1.45	0.22

Q (m^3/s)	V (m/s)	n	α (°)	A (m^2)	Y (m)	B (m)	S	Fr	H (m)	FB (m)
0.25	0.71	0.029	60	0.35	0.45	0.52	0.003	0.39	0.53	0.08
0.50	0.84	0.029	60	0.59	0.58	0.67	0.003	0.41	0.69	0.10
0.75	0.93	0.029	60	0.80	0.68	0.79	0.003	0.42	0.80	0.12
1.00	1.00	0.029	60	1.00	0.76	0.88	0.003	0.43	0.89	0.13
1.50	1.11	0.029	60	1.35	0.88	1.02	0.003	0.44	1.04	0.16
2.00	1.19	0.029	60	1.67	0.98	1.13	0.003	0.44	1.16	0.17
2.50	1.26	0.029	60	1.98	1.07	1.23	0.003	0.45	1.26	0.19
3.00	1.32	0.029	60	2.27	1.14	1.32	0.003	0.46	1.35	0.20

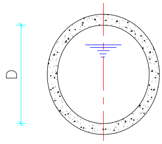
Q (m^3/s)	V (m/s)	n	α (°)	A (m^2)	Y (m)	B (m)	S	Fr	H (m)	FB (m)
0.25	0.79	0.029	60	0.32	0.43	0.49	0.004	0.45	0.50	0.08
0.50	0.94	0.029	60	0.53	0.55	0.64	0.004	0.47	0.65	0.10
0.75	1.04	0.029	60	0.72	0.65	0.74	0.004	0.48	0.76	0.11
1.00	1.12	0.029	60	0.89	0.72	0.83	0.004	0.49	0.85	0.13
1.50	1.24	0.029	60	1.21	0.84	0.97	0.004	0.50	0.98	0.15
2.00	1.33	0.029	60	1.50	0.93	1.08	0.004	0.51	1.10	0.16
2.50	1.41	0.029	60	1.78	1.01	1.17	0.004	0.52	1.19	0.18
3.00	1.47	0.029	60	2.04	1.08	1.25	0.004	0.52	1.28	0.19

Q: design discharge (m^3/s)
V: velocity (m/s)
n: Manning coefficient
A: flow area (m^2)
Y: water depth (m)
FB: freeboard (m)
B: width (m)
S: slope
Fr: Froude number
H: height of structure (m)
α: canal angle with horizontal (°)

C1.4: Buried pipe

C1.4.1: Diameter of buried pipe as a function of discharge and pipe slope

Q: Design discharge (m^3/s)
D: Buried pipe diameter (m)
S: Buried pipe slope



Values = c * Q ^{d}

Values: diameter [D]

S=0.001

Values	c	d
D	1.1685	0.375

S=0.002

Values	c	d
D	1.0261	0.375

S=0.003

Values	c	d
D	0.9510	0.375

S=0.004

Values	c	d
D	0.9010	0.3750

C1.4.2: Concrete volume, reinforcement, excavation, formwork and backfilling of buried pipe as a function of discharge and pipe slope

Values = c * Q ^{d}

Rocky bed

Values: concrete volume, reinforcment, excavation, formwork and backfilling

S=0.001, concrete [m^3]

Comp.	c	d
Con.	0.6321	0.5743

S=0.002, concrete [m^3]

Comp.	c	d
Con.	0.5181	0.5734

S=0.001, reinf. [kg]

Comp.	c	d
Reinf.	31.6070	0.5743

S=0.002, reinf. [kg]

Comp.	c	d
Reinf.	25.9040	0.5734

S=0.001, exc. [m^3]

Comp.	c	d
Exc.	7.0632	0.4678

S=0.002, exc. [m^3]

Comp.	c	d
Exc.	6.0149	0.4497

S=0.001, formwork [m^2]

Comp.	c	d
Fw	8.3236	0.3602

S=0.002, formwork [m^2]

Comp.	c	d
Fw	7.3473	0.3592

S=0.001, backfill. [m^3]

Comp.	c	d
Bf	5.3782	0.4022

S=0.002, backfill. [m^3]

Comp.	c	d
Bf	4.6838	0.3862

For unit length of buried pipe

C1.4.2: Concrete volume, reinforcement, excavation, formwork and backfilling of buried pipe as a function of discharge and pipe slope

Values = c * Q^d

Rocky bed

Values: concrete volume, reinforcment, excavation, formwork and backfilling

S=0.003, concrete [m^3]

Comp.	c	d
Con.	0.4612	0.5729

S=0.004, concrete [m^3]

Comp.	c	d
Con.	0.4248	0.5725

S=0.002, reinf. [kg]

Comp.	c	d
Reinf.	23.0620	0.5729

S=0.004, reinf. [kg]

Comp.	c	d
Reinf.	21.2380	0.5725

S=0.003, exc. [m^3]

Comp.	c	d
Exc.	5.4913	0.4390

S=0.004, exc. [m^3]

Comp.	c	d
Exc.	5.1544	0.4313

S=0.003, formwork [m^2]

Comp.	c	d
Fw	6.8314	0.3586

S=0.004, formwork [m^2]

Comp.	c	d
Fw	6.4879	0.3581

S=0.002, backfill. [m^3]

Comp.	c	d
Bf	4.3311	0.3769

S=0.004, backfill. [m^3]

Comp.	c	d
Bf	4.1017	0.3703

For unit length of buried pipe

C1.4.3: Hydraulic calculations of buried pipe

Q (m^3/s)	n	S	D (m)	Y/D	Y (m)	A (m^2)	P (m)	R (m)	V (m/s)	Fr
0.25	0.0154	**0.001**	**0.69**	**0.85**	0.59	0.34	1.63	0.21	**0.73**	**0.28**
0.50	0.0154	**0.001**	**0.90**	**0.85**	0.77	0.58	2.11	0.27	**0.87**	**0.29**
0.75	0.0154	**0.001**	**1.05**	**0.85**	0.89	0.78	2.46	0.32	**0.96**	**0.30**
1.00	0.0154	**0.001**	**1.17**	**0.85**	0.99	0.97	2.74	0.35	**1.03**	**0.30**
1.50	0.0154	**0.001**	**1.36**	**0.85**	1.16	1.32	3.19	0.41	**1.14**	**0.31**
2.00	0.0154	**0.001**	**1.52**	**0.85**	1.29	1.63	3.56	0.46	**1.22**	**0.32**
2.50	0.0154	**0.001**	**1.65**	**0.85**	1.40	1.93	3.87	0.50	**1.29**	**0.32**
3.00	0.0154	**0.001**	**1.76**	**0.85**	1.50	2.21	4.14	0.54	**1.35**	**0.33**

Q (m^3/s)	n	S	D (m)	Y/D	Y (m)	A (m^2)	P (m)	R (m)	V (m/s)	Fr
0.25	0.0154	**0.002**	**0.61**	**0.85**	0.52	0.26	1.43	0.19	**0.94**	**0.39**
0.50	0.0154	**0.002**	**0.79**	**0.85**	0.67	0.45	1.86	0.24	**1.12**	**0.40**
0.75	0.0154	**0.002**	**0.92**	**0.85**	0.78	0.60	2.16	0.28	**1.24**	**0.41**
1.00	0.0154	**0.002**	**1.03**	**0.85**	0.87	0.75	2.41	0.31	**1.33**	**0.42**
1.50	0.0154	**0.002**	**1.19**	**0.85**	1.02	1.02	2.80	0.36	**1.48**	**0.43**
2.00	0.0154	**0.002**	**1.33**	**0.85**	1.13	1.26	3.12	0.40	**1.59**	**0.44**
2.50	0.0154	**0.002**	**1.45**	**0.85**	1.23	1.49	3.39	0.44	**1.68**	**0.45**
3.00	0.0154	**0.002**	**1.55**	**0.85**	1.32	1.71	3.63	0.47	**1.76**	**0.45**

Q (m^3/s)	n	S	D (m)	Y/D	Y (m)	A (m^2)	P (m)	R (m)	V (m/s)	Fr
0.25	0.0154	**0.003**	**0.57**	**0.85**	0.48	0.23	1.33	0.17	**1.10**	**0.47**
0.50	0.0154	**0.003**	**0.73**	**0.85**	0.62	0.38	1.72	0.22	**1.31**	**0.49**
0.75	0.0154	**0.003**	**0.85**	**0.85**	0.73	0.52	2.00	0.26	**1.45**	**0.50**
1.00	0.0154	**0.003**	**0.95**	**0.85**	0.81	0.64	2.23	0.29	**1.55**	**0.51**
1.50	0.0154	**0.003**	**1.11**	**0.85**	0.94	0.87	2.60	0.34	**1.72**	**0.52**
2.00	0.0154	**0.003**	**1.23**	**0.85**	1.05	1.08	2.89	0.37	**1.85**	**0.53**
2.50	0.0154	**0.003**	**1.34**	**0.85**	1.14	1.28	3.15	0.41	**1.95**	**0.54**
3.00	0.0154	**0.003**	**1.44**	**0.85**	1.22	1.47	3.37	0.44	**2.05**	**0.55**

Q (m^3/s)	n	S	D (m)	Y/D	Y (m)	A (m^2)	P (m)	R (m)	V (m/s)	Fr
0.25	0.0154	**0.004**	**0.54**	**0.85**	0.46	0.20	1.26	0.16	**1.22**	**0.53**
0.50	0.0154	**0.004**	**0.69**	**0.85**	0.59	0.34	1.63	0.21	**1.46**	**0.56**
0.75	0.0154	**0.004**	**0.81**	**0.85**	0.69	0.47	1.90	0.25	**1.61**	**0.57**
1.00	0.0154	**0.004**	**0.90**	**0.85**	0.77	0.58	2.11	0.27	**1.73**	**0.58**
1.50	0.0154	**0.004**	**1.05**	**0.85**	0.89	0.78	2.46	0.32	**1.92**	**0.60**
2.00	0.0154	**0.004**	**1.17**	**0.85**	0.99	0.97	2.74	0.35	**2.06**	**0.61**
2.50	0.0154	**0.004**	**1.27**	**0.85**	1.08	1.15	2.98	0.39	**2.18**	**0.62**
3.00	0.0154	**0.004**	**1.36**	**0.85**	1.16	1.32	3.19	0.41	**2.28**	**0.62**

Q: design discharge (m^3/s)

V: velocity (m/s)

n: Manning coefficient

A: flow area (m^2)

Y: water depth (m)

R: hydraulic radius (m)

P: wet perimeter (m)

S: slope

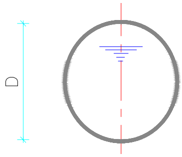
Fr: Froude number

D: diameter of structure (m)

C1.5: Buried PVC pipe

C1.5.1: Diameter of buried PVC pipe as a function of discharge and pipe slope

Q: Design discharge (m^3/s)
D: Buried PVC pipe diameter (m)
S: Buried PVC pipe slope



Values = c * Q ^{d}

Values: diameter [D]

S=0.001

Values	c	d
D	1.0567	0.375

S=0.002

Values	c	d
D	0.9279	0.375

S=0.003

Values	c	d
D	0.860	0.375

S=0.004

Values	c	d
D	0.8148	0.375

C1.5.2: Excavation and backfilling of buried PVC pipe as a function of discharge and pipe slope

Values = c * Q ^{d}

Rocky bed

Values: excavation and backfilling

S=0.001, excavation [m^3]

Comp.	c	d
Exc.	6.2245	0.3817

S=0.002, excavation [m^3]

Comp.	c	d
Exc.	5.4629	0.3578

S=0.001, backfilling [m^3]

Comp.	c	d
Bf	5.1911	0.3141

S=0.002, backfilling [m^3]

Comp.	c	d
Bf	4.6617	0.2958

S=0.003, excavation [m^3]

Comp.	c	d
Exc.	5.0806	0.3438

S=0.004, excavation [m^3]

Comp.	c	d
Exc.	4.8339	0.3340

S=0.003, backfilling [m^3]

Comp.	c	d
Bf	4.3902	0.2853

S=0.004, backfilling [m^3]

Comp.	c	d
Bf	4.2127	0.2778

For unit length of buried PVC pipe

C1.5.3: Hydraulic calculations of buried PVC pipe

Q (m^3/s)	n	S	D (m)	Y/D	Y (m)	A (m^2)	P (m)	R (m)	V (m/s)	Fr
0.25	0.0118	**0.001**	**0.63**	**0.85**	0.53	0.28	1.47	0.19	**0.89**	**0.36**
0.50	0.0118	**0.001**	**0.81**	**0.85**	0.69	0.47	1.91	0.25	**1.06**	**0.38**
0.75	0.0118	**0.001**	**0.95**	**0.85**	0.81	0.64	2.23	0.29	**1.17**	**0.38**
1.00	0.0118	**0.001**	**1.06**	**0.85**	0.90	0.79	2.48	0.32	**1.26**	**0.39**
1.50	0.0118	**0.001**	**1.23**	**0.85**	1.05	1.08	2.89	0.37	**1.39**	**0.40**
2.00	0.0118	**0.001**	**1.37**	**0.85**	1.16	1.34	3.22	0.42	**1.50**	**0.41**
2.50	0.0118	**0.001**	**1.49**	**0.85**	1.27	1.58	3.50	0.45	**1.58**	**0.41**
3.00	0.0118	**0.001**	**1.60**	**0.85**	1.36	1.81	3.74	0.48	**1.66**	**0.42**

Q (m^3/s)	n	S	D (m)	Y/D	Y (m)	A (m^2)	P (m)	R (m)	V (m/s)	Fr
0.25	0.0118	**0.002**	**0.55**	**0.85**	0.47	0.22	1.29	0.17	**1.15**	**0.50**
0.50	0.0118	**0.002**	**0.72**	**0.85**	0.61	0.36	1.68	0.22	**1.37**	**0.52**
0.75	0.0118	**0.002**	**0.83**	**0.85**	0.71	0.49	1.95	0.25	**1.52**	**0.53**
1.00	0.0118	**0.002**	**0.93**	**0.85**	0.79	0.61	2.18	0.28	**1.63**	**0.54**
1.50	0.0118	**0.002**	**1.08**	**0.85**	0.92	0.83	2.53	0.33	**1.81**	**0.56**
2.00	0.0118	**0.002**	**1.20**	**0.85**	1.02	1.03	2.82	0.36	**1.94**	**0.57**
2.50	0.0118	**0.002**	**1.31**	**0.85**	1.11	1.22	3.07	0.40	**2.05**	**0.57**
3.00	0.0118	**0.002**	**1.40**	**0.85**	1.19	1.40	3.29	0.42	**2.15**	**0.58**

Q (m^3/s)	n	S	D (m)	Y/D	Y (m)	A (m^2)	P (m)	R (m)	V (m/s)	Fr
0.25	0.0118	**0.003**	**0.51**	**0.85**	0.43	0.19	1.20	0.16	**1.34**	**0.60**
0.50	0.0118	**0.003**	**0.66**	**0.85**	0.56	0.31	1.56	0.20	**1.60**	**0.63**
0.75	0.0118	**0.003**	**0.77**	**0.85**	0.66	0.42	1.81	0.23	**1.77**	**0.64**
1.00	0.0118	**0.003**	**0.86**	**0.85**	0.73	0.53	2.02	0.26	**1.90**	**0.66**
1.50	0.0118	**0.003**	**1.00**	**0.85**	0.85	0.71	2.35	0.30	**2.10**	**0.67**
2.00	0.0118	**0.003**	**1.12**	**0.85**	0.95	0.88	2.62	0.34	**2.26**	**0.68**
2.50	0.0118	**0.003**	**1.21**	**0.85**	1.03	1.05	2.84	0.37	**2.39**	**0.69**
3.00	0.0118	**0.003**	**1.30**	**0.85**	1.10	1.20	3.05	0.39	**2.50**	**0.70**

Q (m^3/s)	n	S	D (m)	Y/D	Y (m)	A (m^2)	P (m)	R (m)	V (m/s)	Fr
0.25	0.0118	**0.004**	**0.48**	**0.85**	0.41	0.17	1.14	0.15	**1.50**	**0.69**
0.50	0.0118	**0.004**	**0.63**	**0.85**	0.53	0.28	1.47	0.19	**1.78**	**0.72**
0.75	0.0118	**0.004**	**0.73**	**0.85**	0.62	0.38	1.72	0.22	**1.97**	**0.74**
1.00	0.0118	**0.004**	**0.81**	**0.85**	0.69	0.47	1.91	0.25	**2.12**	**0.75**
1.50	0.0118	**0.004**	**0.95**	**0.85**	0.81	0.64	2.23	0.29	**2.34**	**0.77**
2.00	0.0118	**0.004**	**1.06**	**0.85**	0.90	0.79	2.48	0.32	**2.52**	**0.78**
2.50	0.0118	**0.004**	**1.15**	**0.85**	0.98	0.94	2.70	0.35	**2.66**	**0.79**
3.00	0.0118	**0.004**	**1.23**	**0.85**	1.05	1.08	2.89	0.37	**2.79**	**0.80**

C1.6: Buried pipe (under pressure)

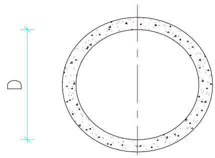
C1.6.1: Diameter of buried pipe (under pressure) as a function of discharge

Q: Design discharge (m^3/s)

D: Berid pipe diameter

S: Buried pipe slope

Pressurized flow



Values = c * Q ^{d}

Values	c	d
D	1.1119	0.3767

Values: diameter [D]

C1.6.2: Concrete volume, reinforcement, excavation, formwork and backfilling of buried pipe (under pressure) as a function of discharge

Values: concrete volume, reinforcment, excavation, formwork and backfilling

Rocky bed

Concrete volume [m^3]

Component	c	d
Con.	0.6126	0.5747

Reinforcement [kg]

Component	c	d
Reinf.	30.6290	0.5747

Excavation [m^3]

Component	c	d
Exc.	6.7110	0.4625

Formwork [m^2]

Component	c	d
Fw	7.9753	0.3608

Backfilling [m^3]

Component	c	d
Bf	5.1465	0.3975

For unit length of buried pipe

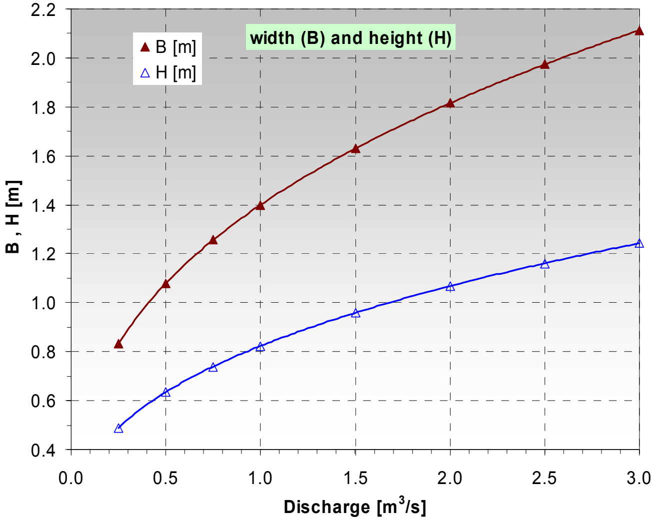
C1.6.3: Hydraulic calculations of buried pipe (under pressure)

D [m]	A [m^2]	Q [m^3/s]	V [m/s]
0.54	0.23	**0.25**	**1.10**
0.70	0.38	**0.50**	**1.31**
0.81	0.52	**0.75**	**1.45**
0.91	0.64	**1.00**	**1.55**
1.05	0.87	**1.50**	**1.72**
1.17	1.08	**2.00**	**1.85**
1.28	1.28	**2.50**	**1.95**
1.37	1.47	**3.00**	**2.05**

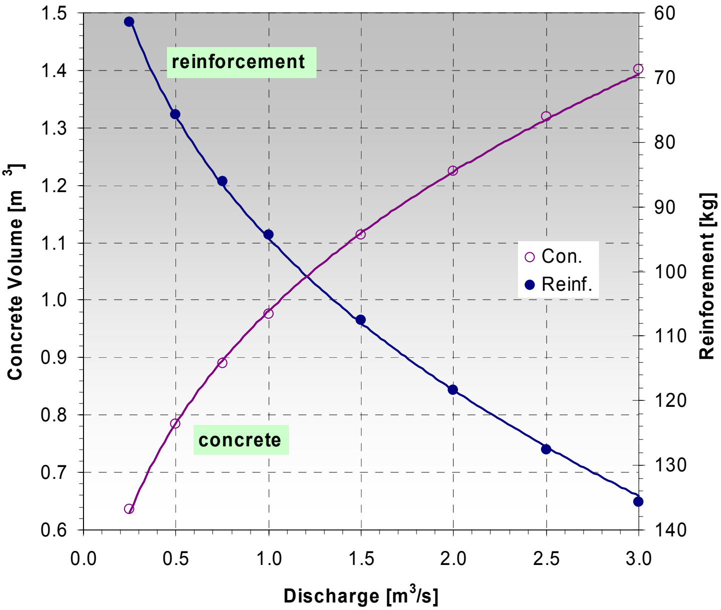
C2: Design charts of headrace canal

C2.1: Open rectangular canal

C2.1.1: Width and height of open canal as a function of discharge and for S=0.001

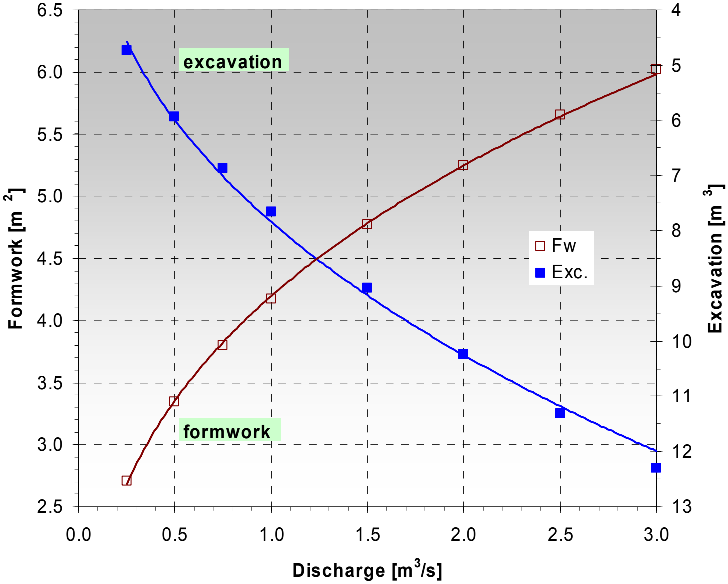


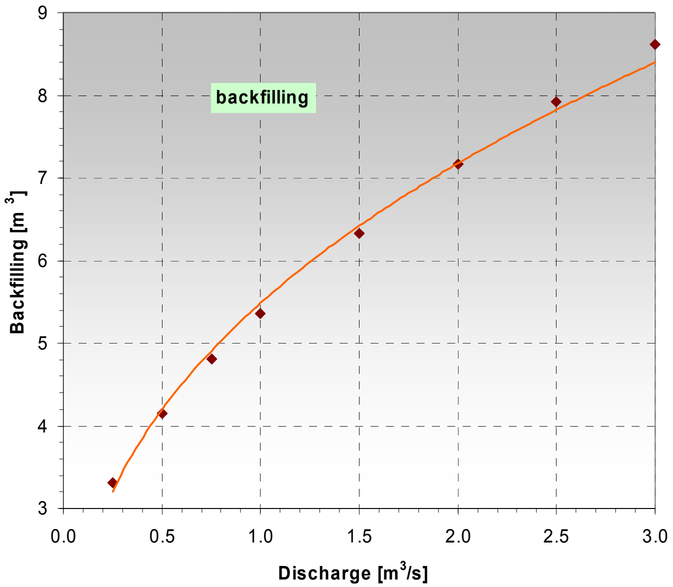
C2.1.2: Concrete volume, reinforcement, excavation, formwork and backfilling of open canal as a function of discharge and for S=0.001



For unit length of open canal

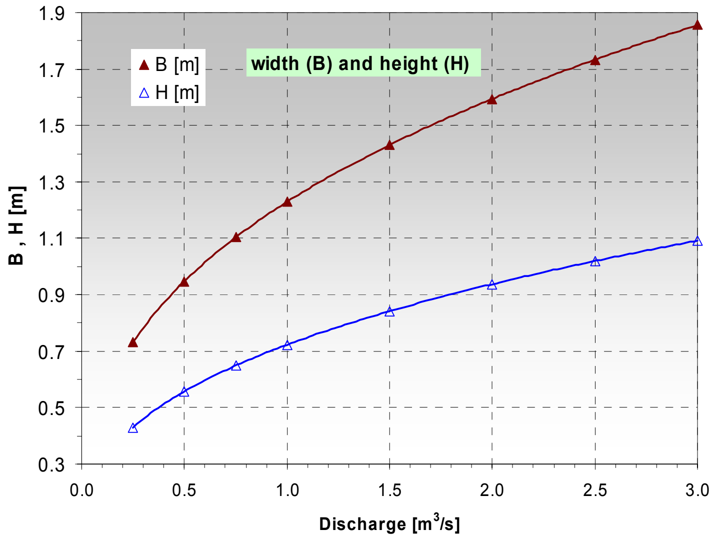
C2.1.2: Concrete volume, reinforcement, excavation, formwork and backfilling of open canal as a function of discharge and for S=0.001



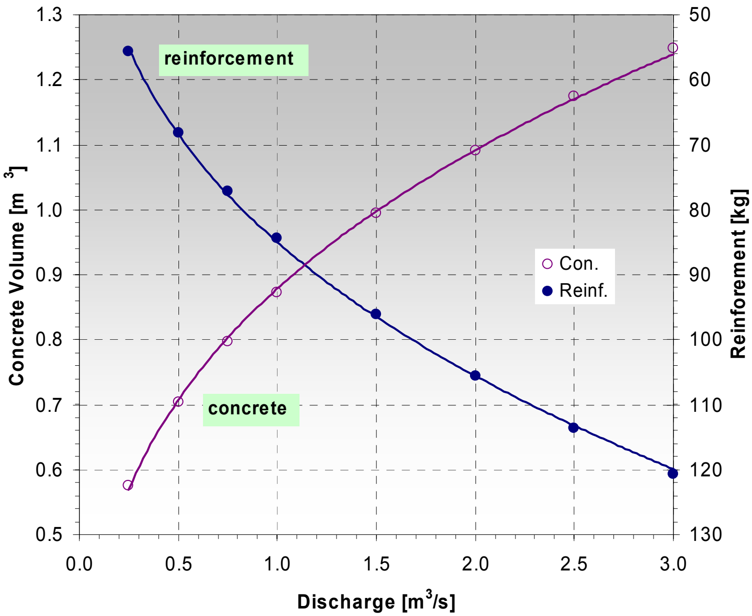


For unit length of open canal

C2.1.3: Width and height of open canal as a function of discharge and for S=0.002

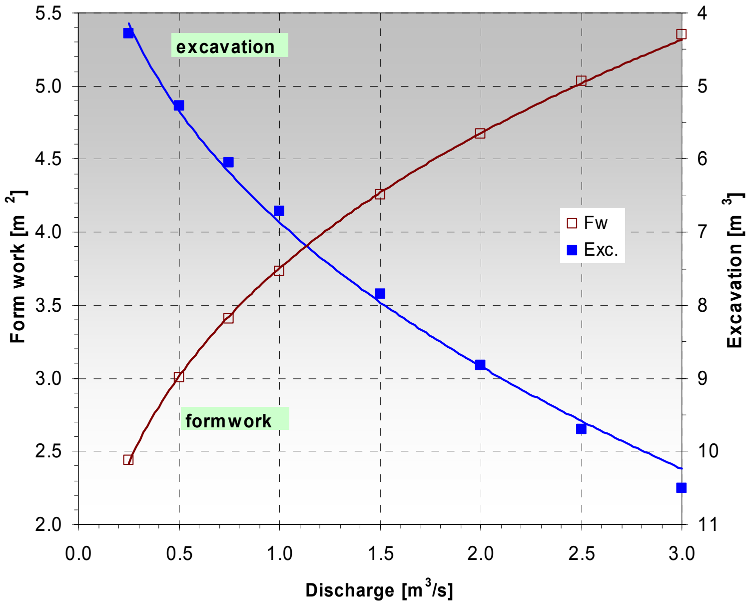


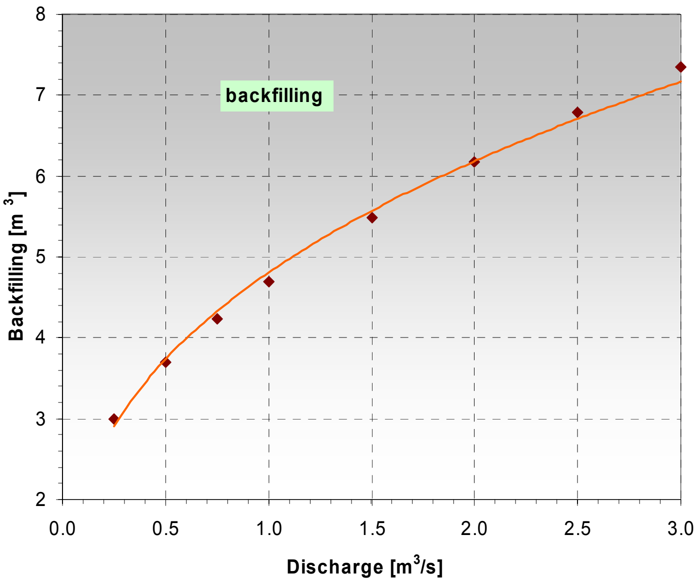
C2.1.4: Concrete volume, reinforcement, excavation, formwork and backfilling of open canal as a function of discharge and for S=0.002



For unit length of open canal

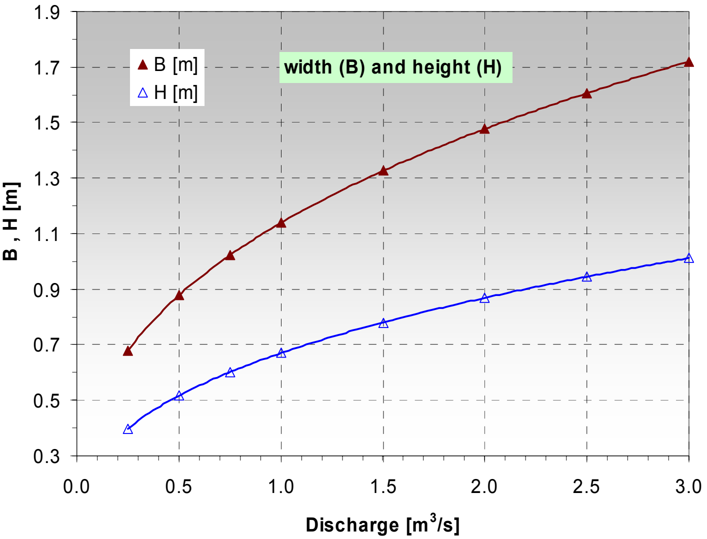
C2.1.4: Concrete volume, reinforcement, excavation, formwork and backfilling of open canal as a function of discharge and for S=0.002



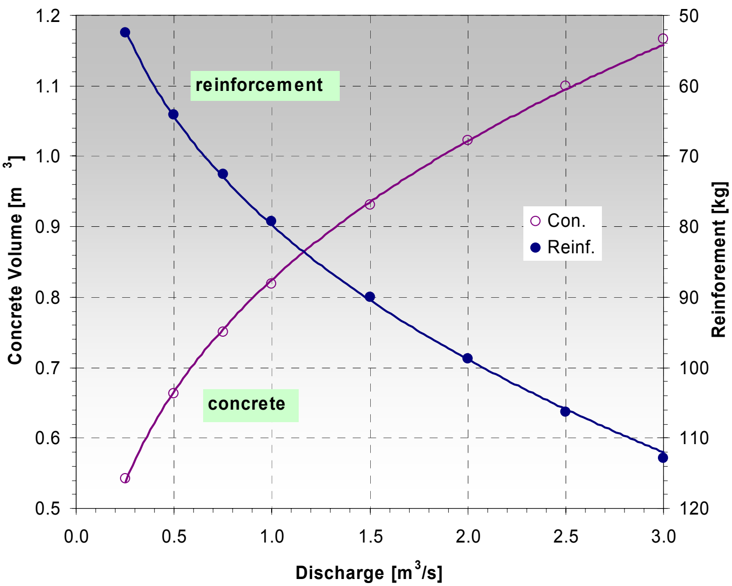


For unit length of open canal

C2.1.5: Width and height of open canal as a function of discharge and for S=0.003

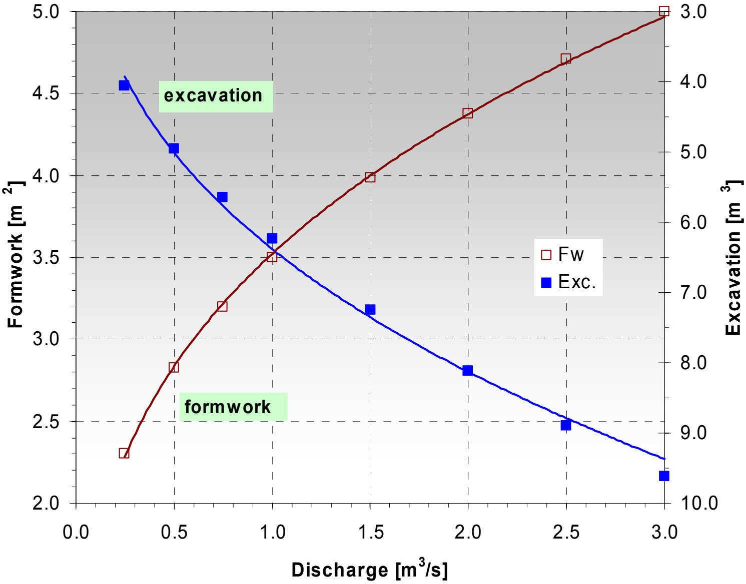


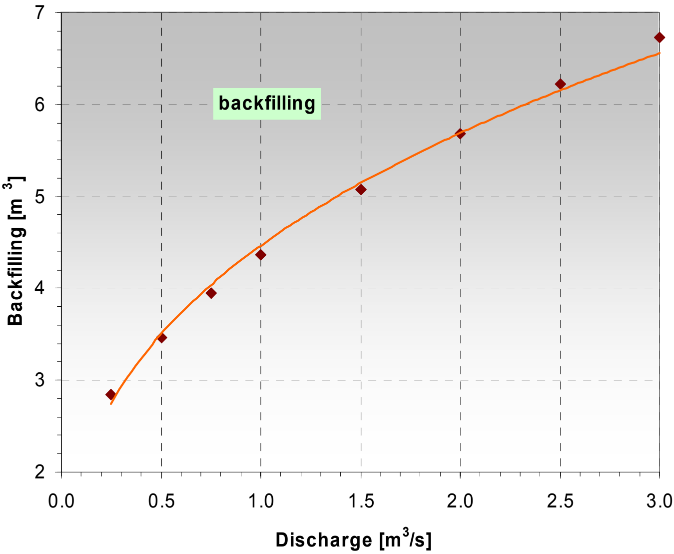
C2.1.6: Concrete volume, reinforcement, excavation, formwork and backfilling of open canal as a function of discharge and for S=0.003



For unit length of open canal

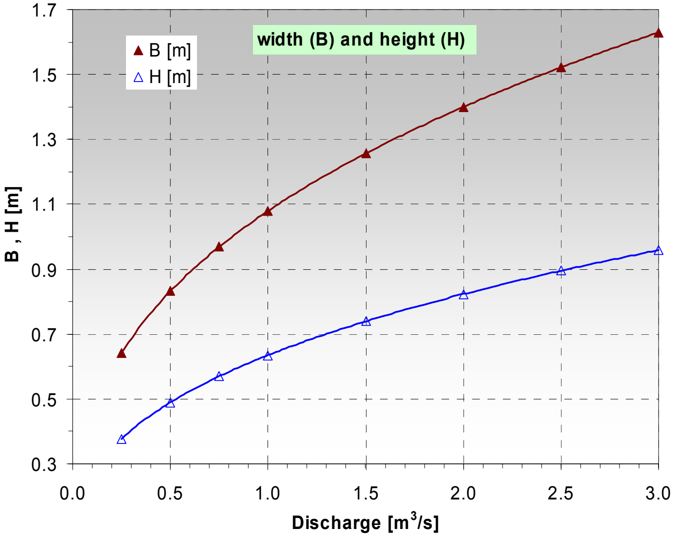
C2.1.6: Concrete volume, reinforcement, excavation, formwork and backfilling of open canal as a function of discharge and for S=0.003



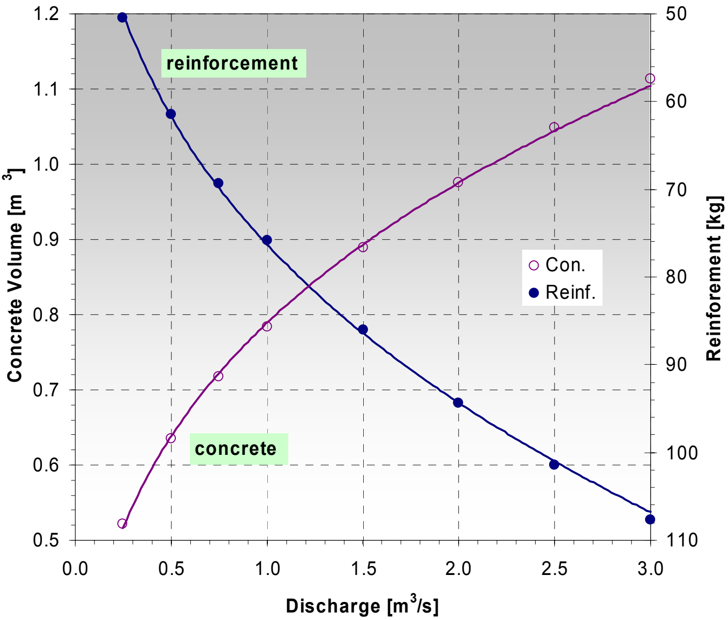


For unit length of open canal

C2.1.7: Width and height of open canal as a function of discharge and for S=0.004

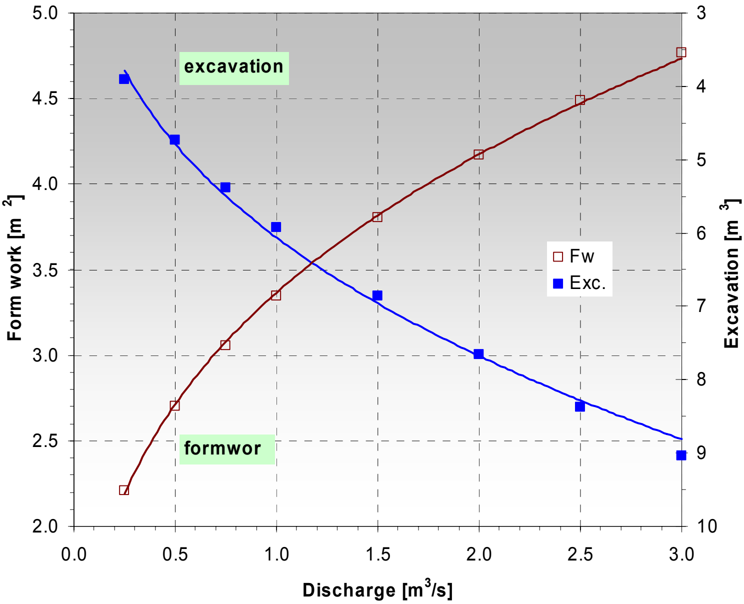


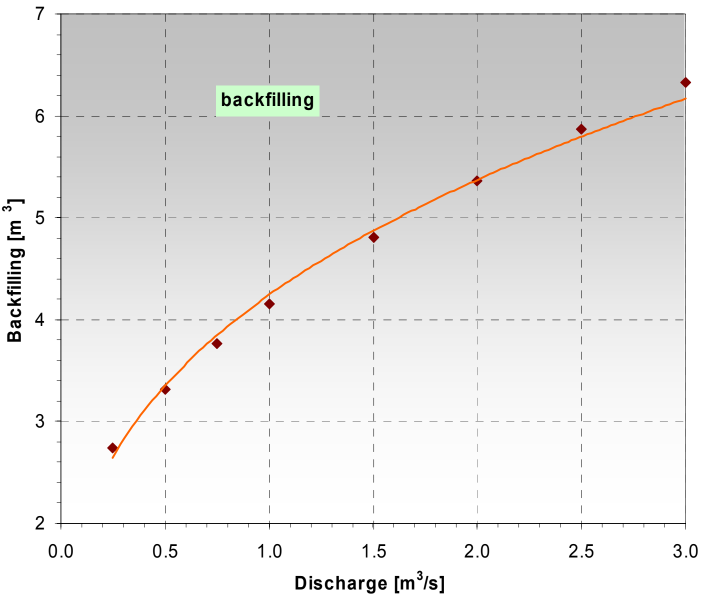
C2.1.8 Concrete volume, reinforcement, excavation, formwork and backfilling of open canal as a function of discharge and for S=0.004



For unit length of open canal

C2.1.8 Concrete volume, reinforcement, excavation, formwork and backfilling of open canal as a function of discharge and for S=0.004

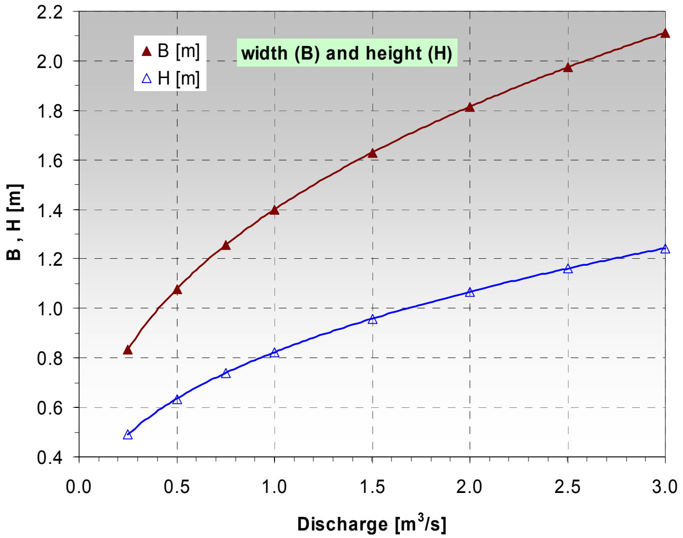




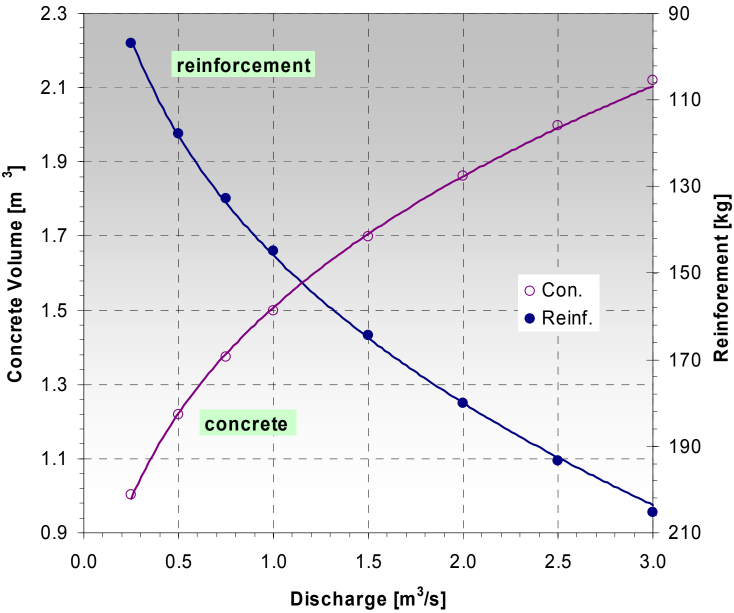
For unit length of open canal

C2.2: Buried rectangular canal

C2.2.1: Width and height of buried canal as a function of discharge and for S=0.001

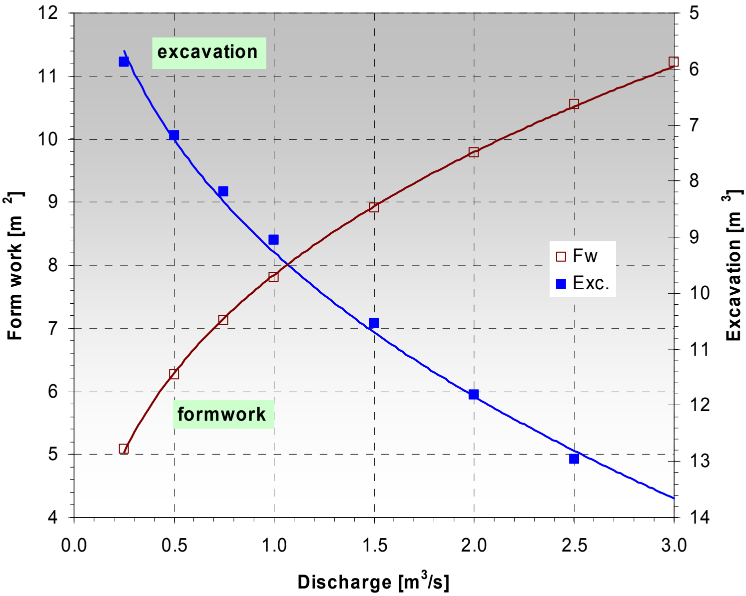


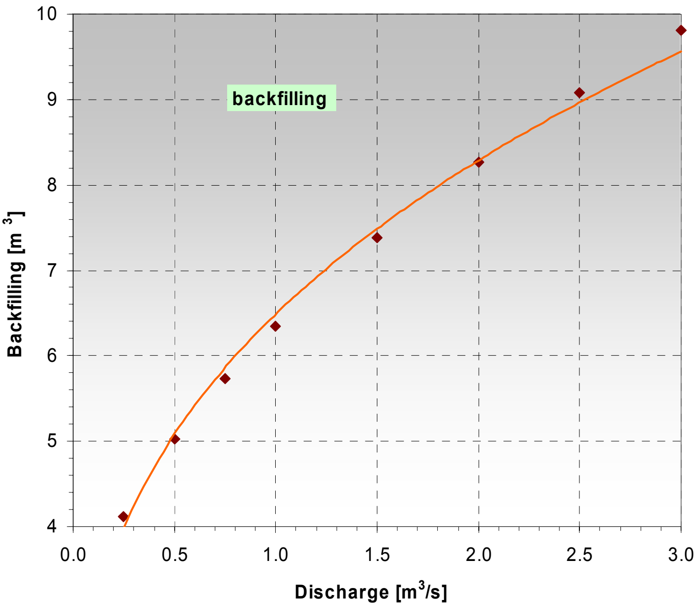
C2.2.2: Concrete volume, reinforcement, excavation, formwork and backfilling of buried canal as a function of discharge and for S=0.001



For unit length of buried canal

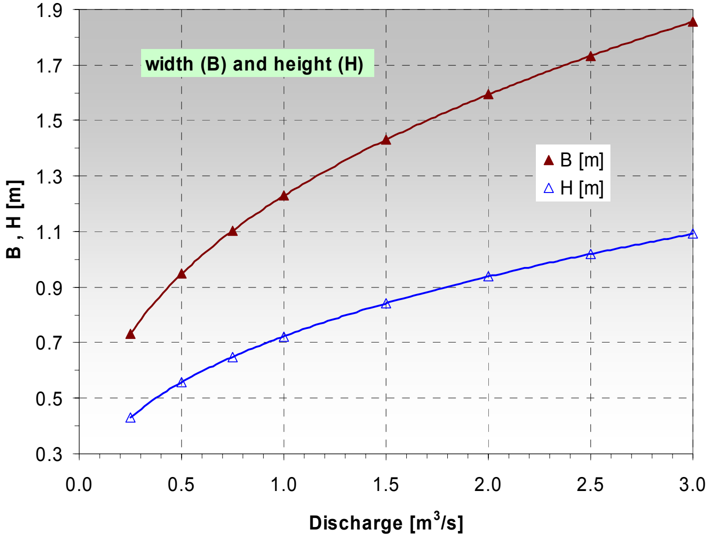
C2.2.2: Concrete volume, reinforcement, excavation, formwork and backfilling of buried canal as a function of discharge and for S=0.001



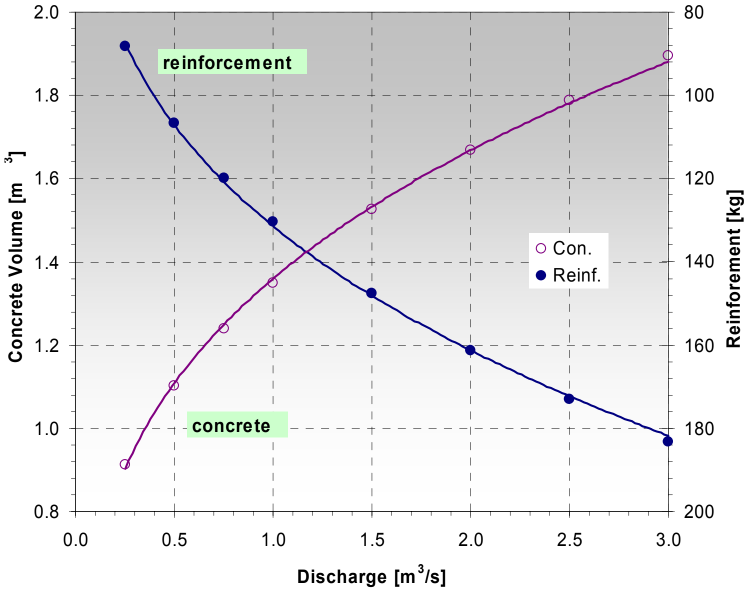


For unit length of buried canal

C2.2.3: Width and height of buried canal as a function of discharge and for S=0.002

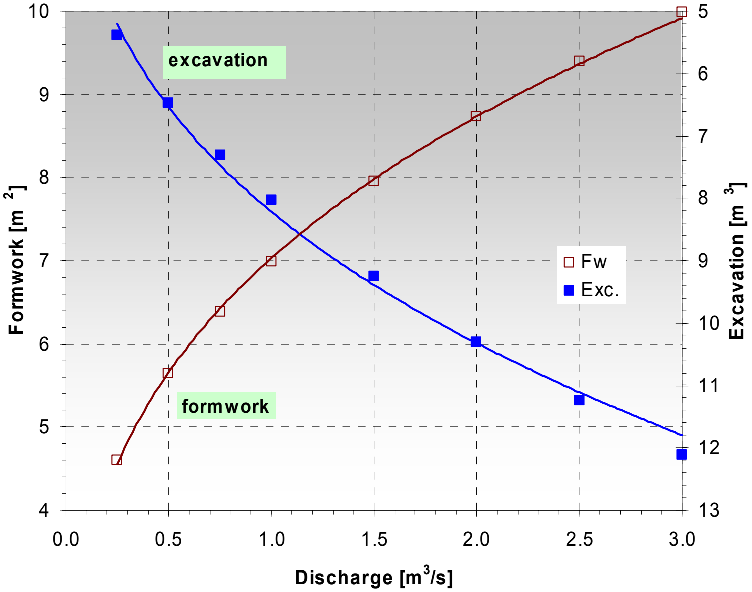


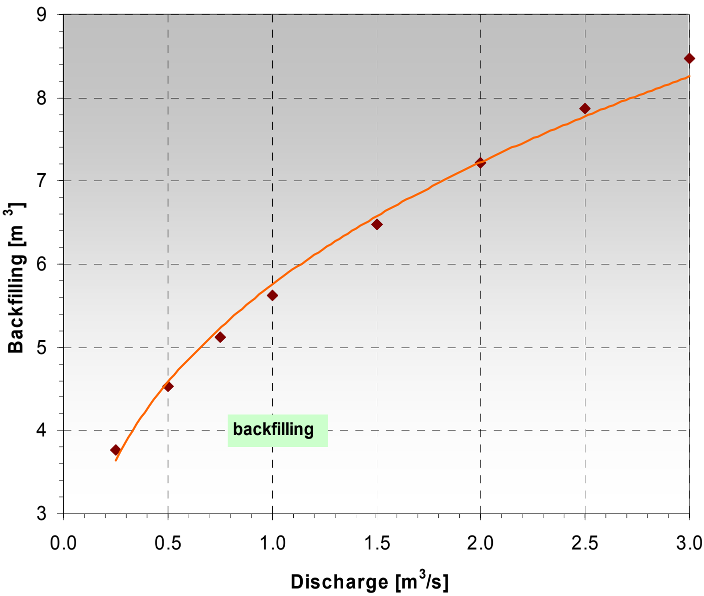
C2.2.4: Concrete volume, reinforcement, excavation, formwork and backfilling of buried canal as a function of discharge and for S=0.002



For unit length of buried canal

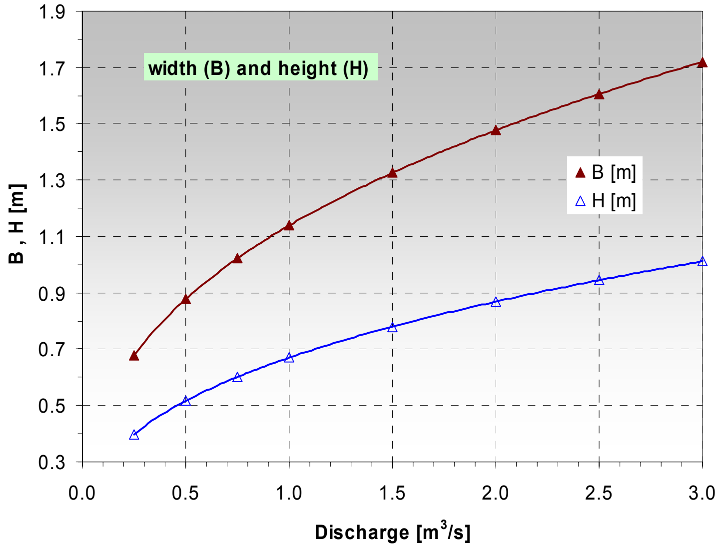
C2.2.4: Concrete volume, reinforcement, excavation, formwork and backfilling of buried canal as a function of discharge and for S=0.002



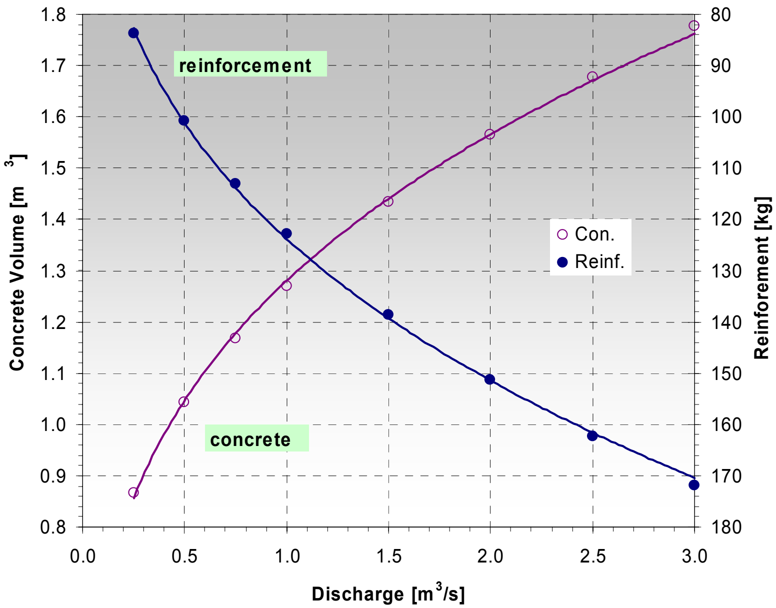


For unit length of buried canal

C2.2.5: Width and height of buried canal as a function of discharge and for S=0.003

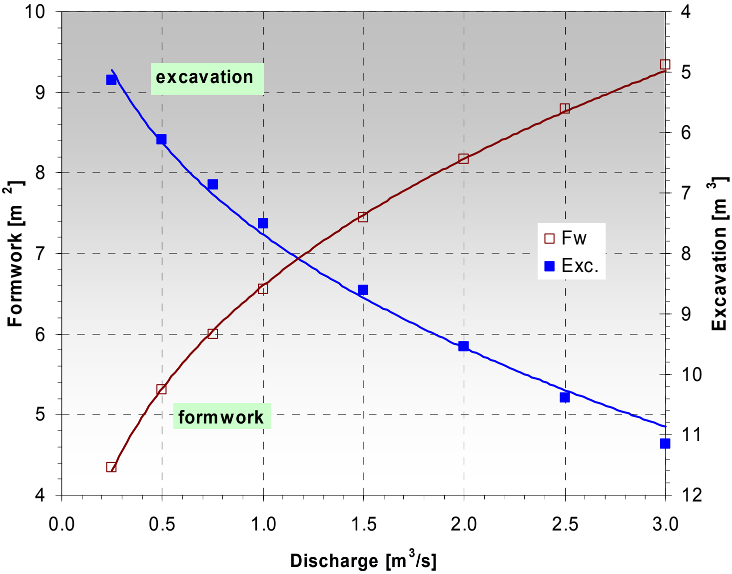


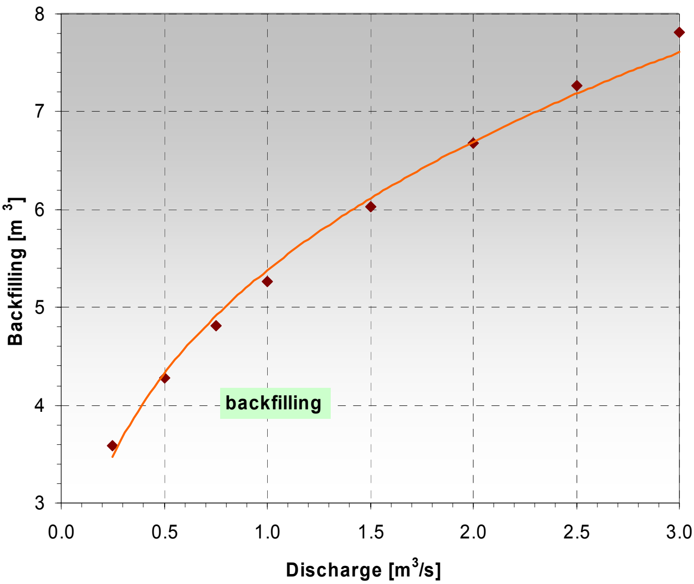
C2.2.6: Concrete volume, reinforcement, excavation, formwork and backfilling of buried canal as a function of discharge and for S=0.003



For unit length of buried canal

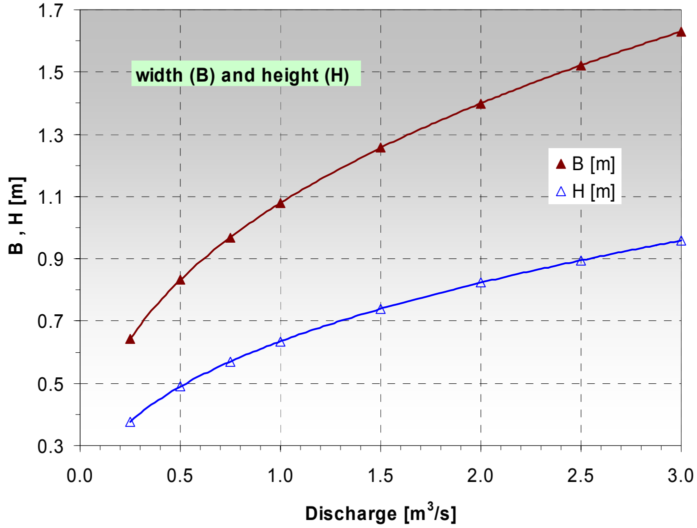
C2.2.6: Concrete volume, reinforcement, excavation, formwork and backfilling of buried canal as a function of discharge and for S=0.003



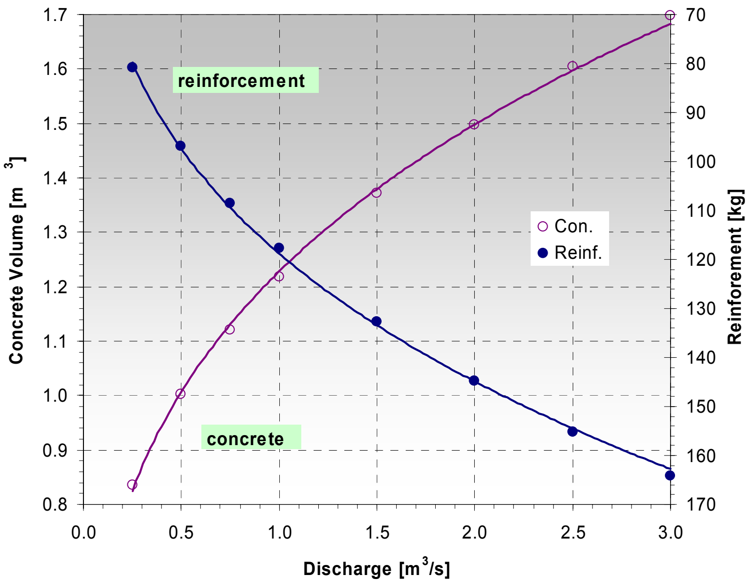


For unit length of buried canal

C2.2.7: Width and height of buried canal as a function of discharge and for S=0.004

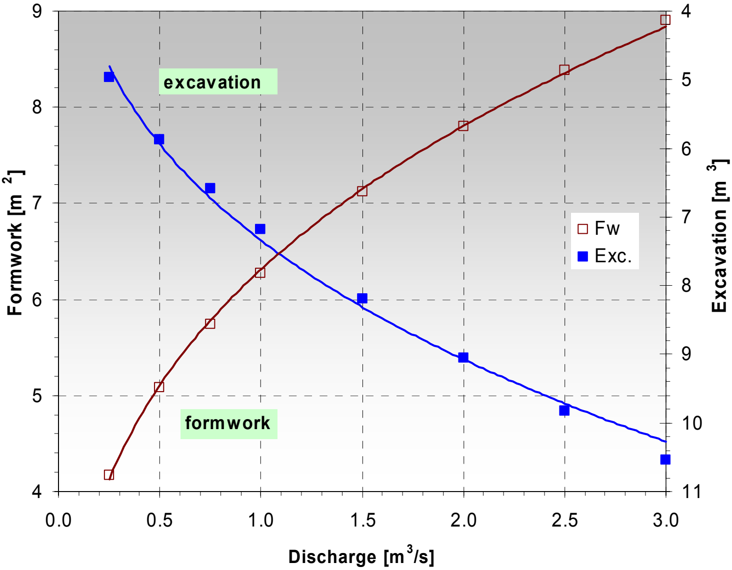


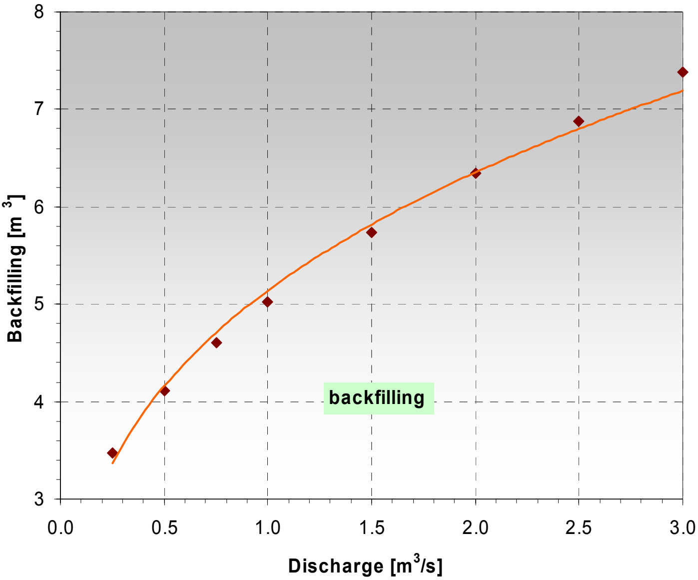
C2.2.8: Concrete volume, reinforcement, excavation, formwork and backfilling of buried canal as a function of discharge and for S=0.004



For unit length of buried canal

C2.2.8: Concrete volume, reinforcement, excavation, formwork and backfilling of buried

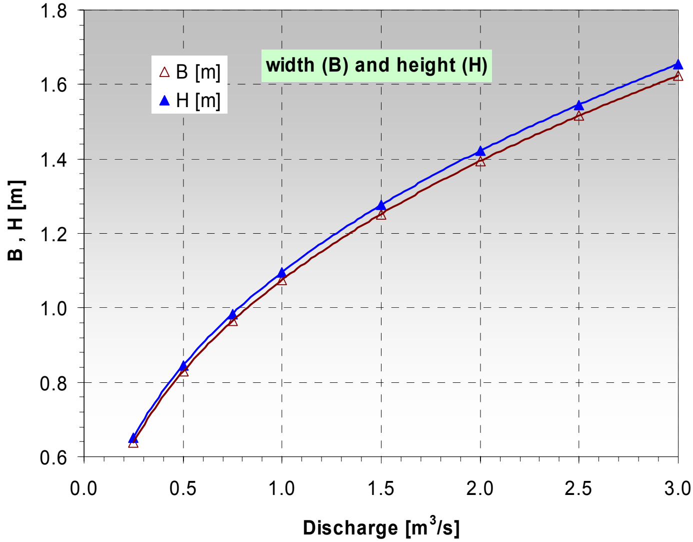




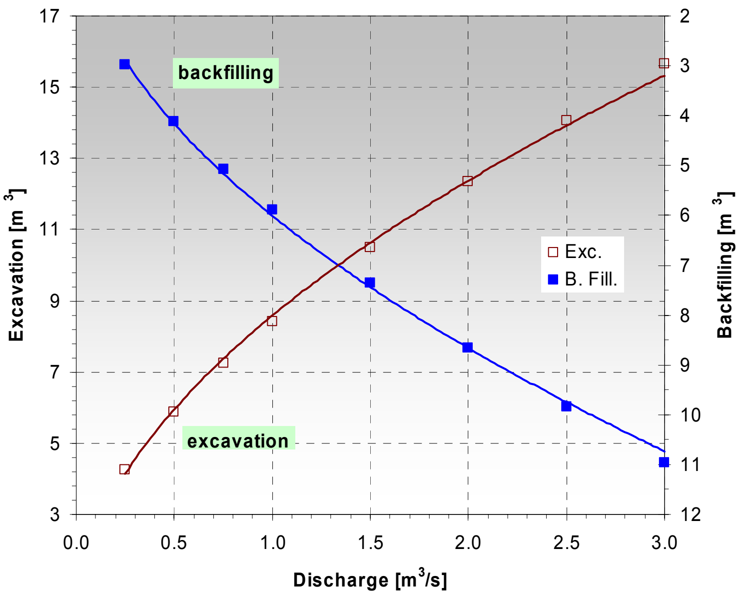
For unit length of buried canal

C2.3: Open rock canal

C2.3.1: Width and height of open rock canal as a function of discharge and for S=0.001

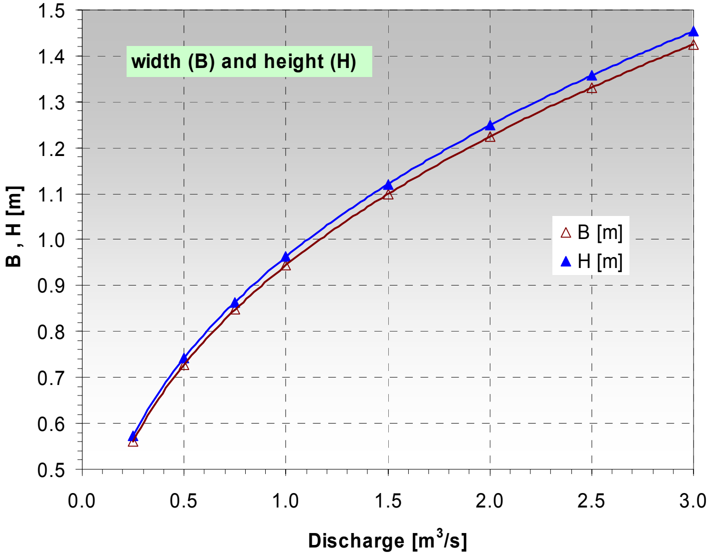


C2.3.2: Excavation and backfilling of open rock canal as a function of discharge and for S=0.001

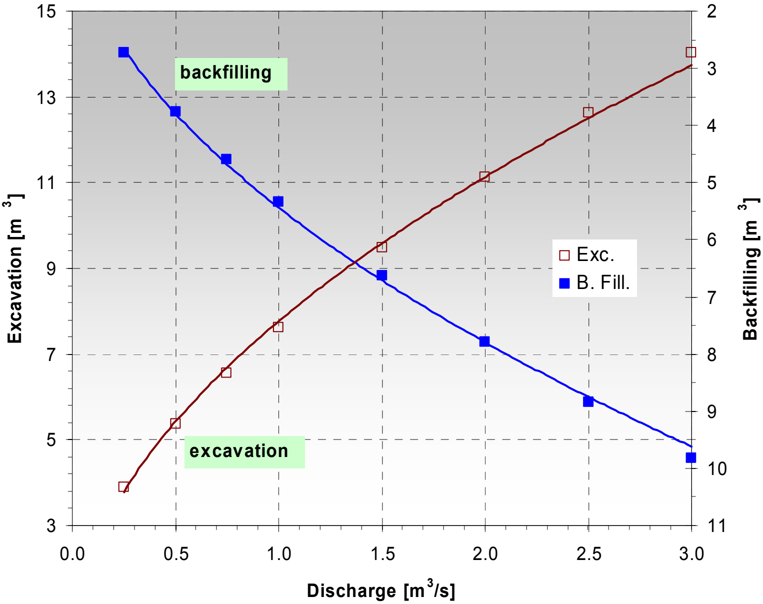


For unit length of open rock canal

C2.3.3: Width and height of open rock canal as a function of discharge and for S=0.002

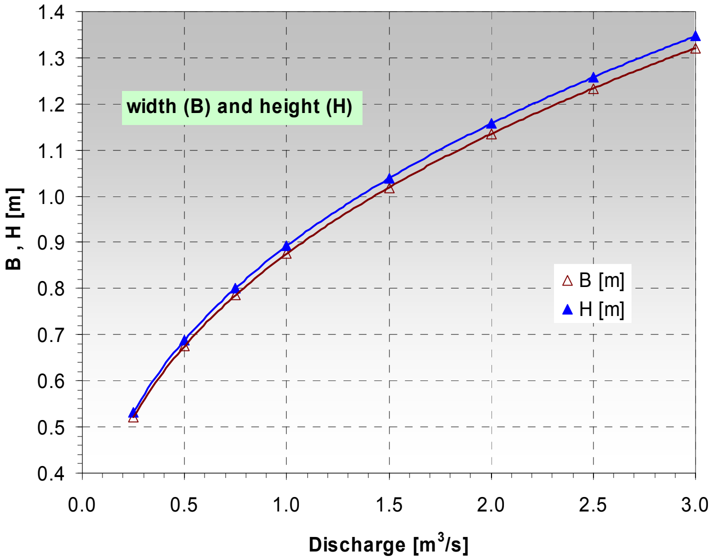


C2.3.4: Excavation and backfilling of open rock canal as a function of discharge and for S=0.002

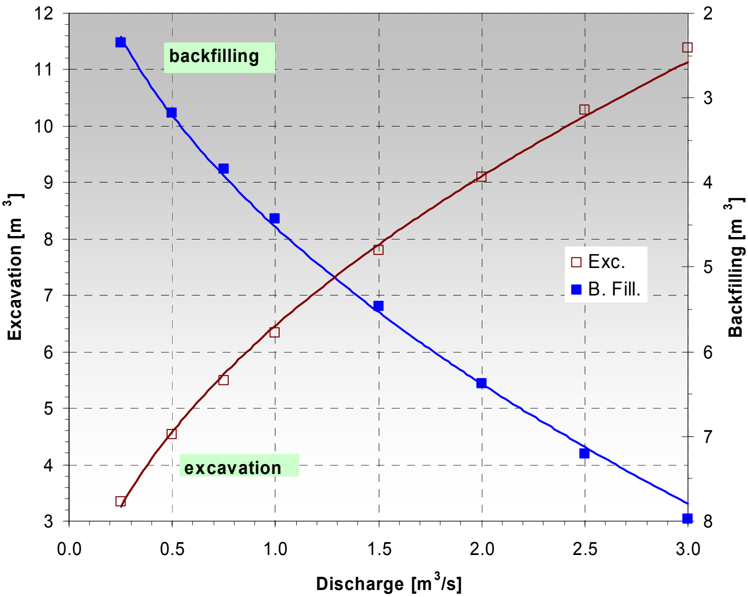


For unit length of open rock canal

C2.3.5: Width and height of open rock canal as a function of discharge and for S=0.003

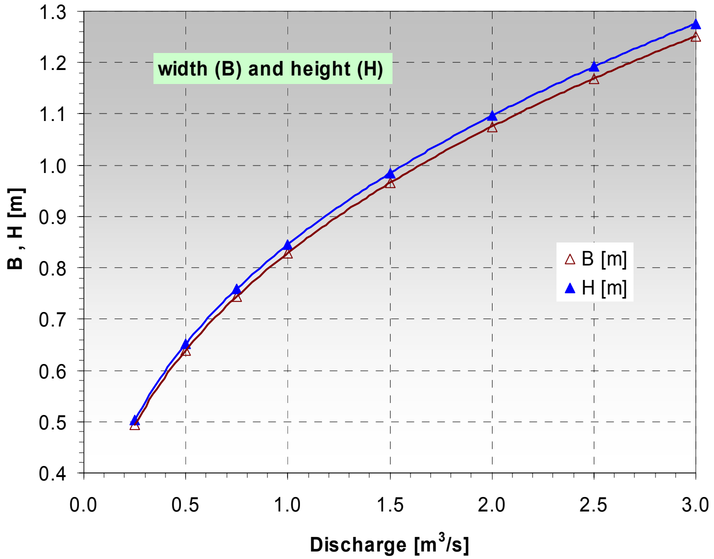


C2.3.6: Excavation and backfilling of open rock canal as a function of discharge and for S=0.003

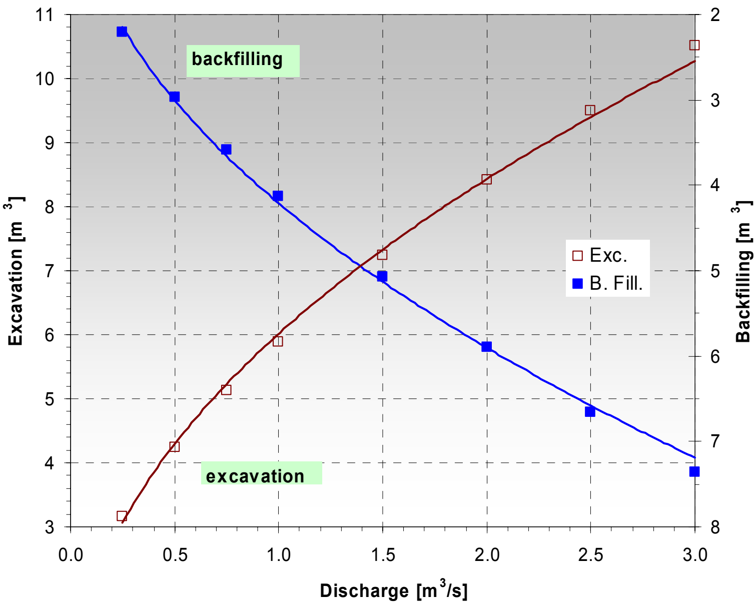


For unit length of open rock canal

C2.3.7: Width and height of open rock canal as a function of discharge and for S=0.004



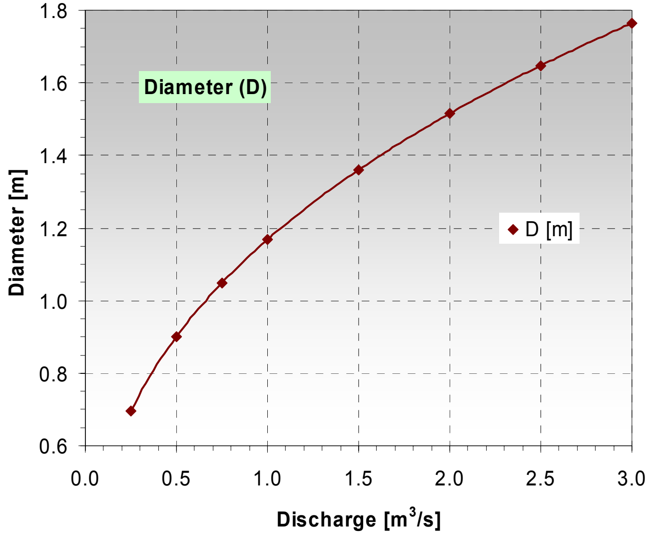
C2.3.8: Excavation and backfilling of open rock canal as a function of discharge and for S=0.004



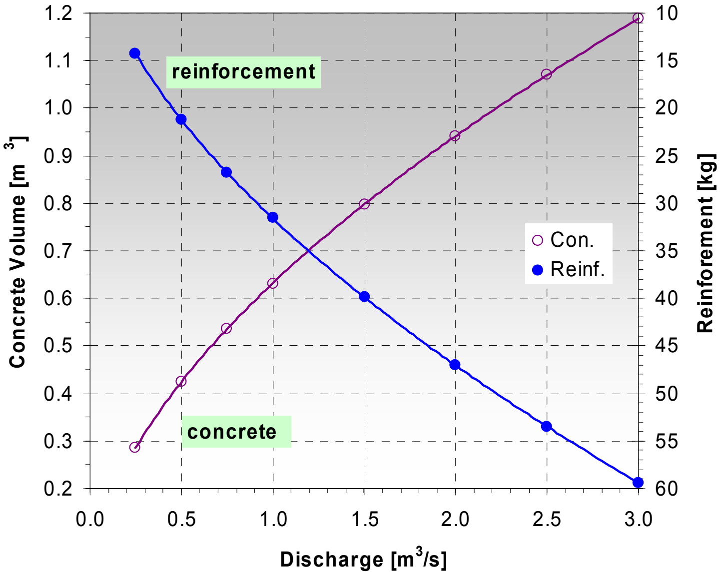
For unit length of open rock canal

C2.4: Buried pipe

C2.4.1: Diameter of buried pipe as a function of discharge and for S=0.001

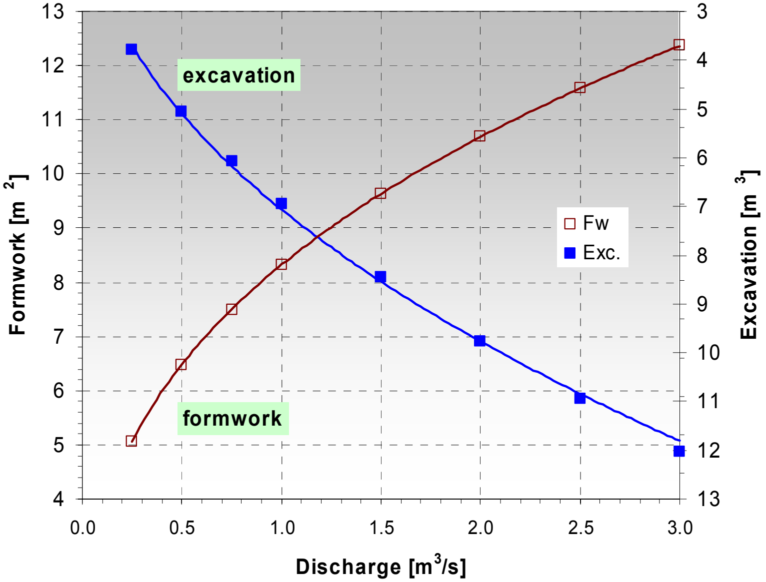


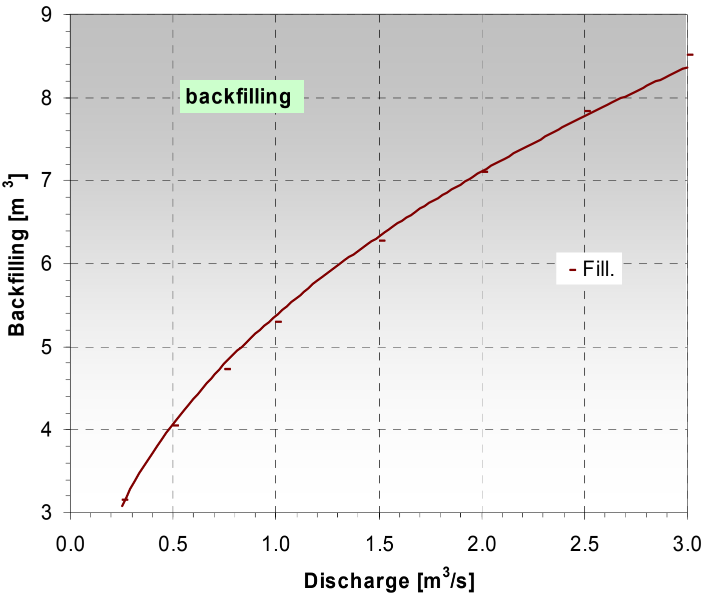
C2.4.2: Concrete volume, reinforcement, excavation, formwork and backfilling of buried pipe as a function of discharge and for S=0.001



For unit length of buried pipe

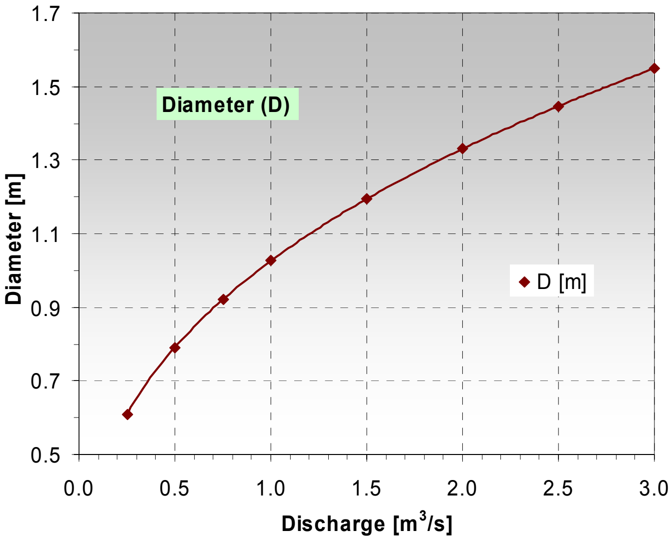
C2.4.2: Concrete volume, reinforcement, excavation, formwork and backfilling of buried pipe as a function of discharge and for S=0.001



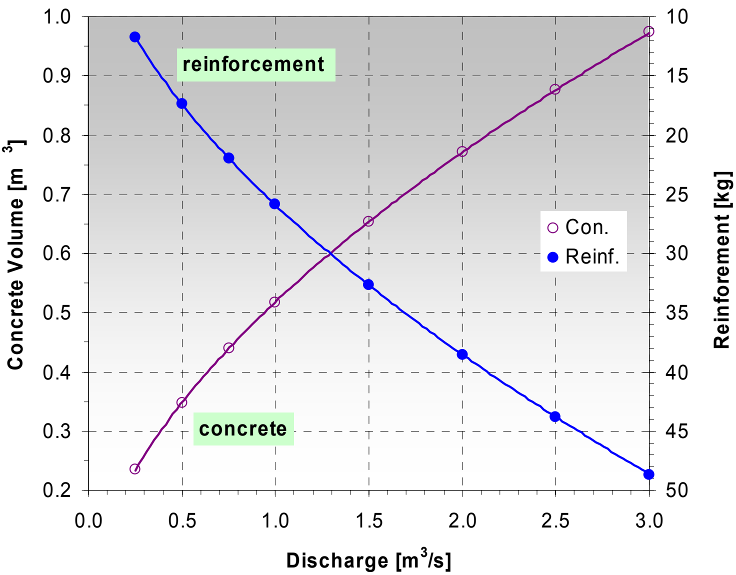


For unit length of buried pipe

C2.4.3: Diameter of buried pipe as a function of discharge and for S=0.002

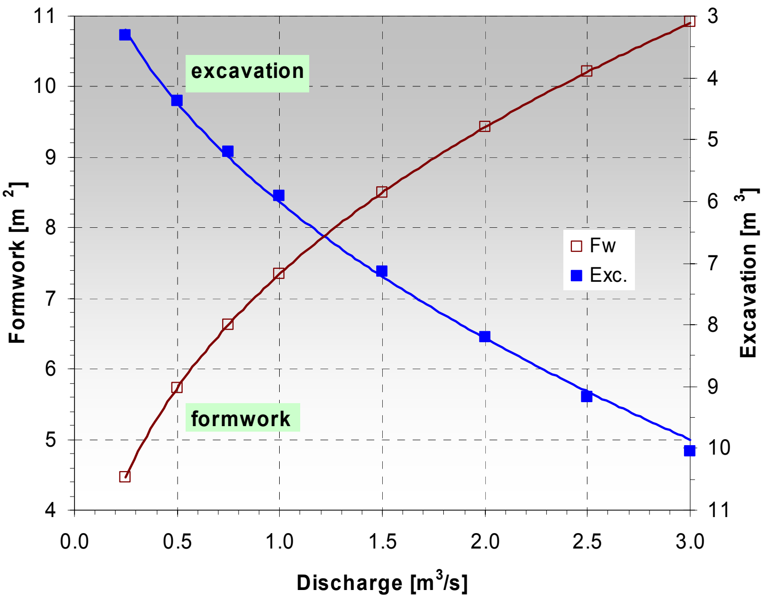


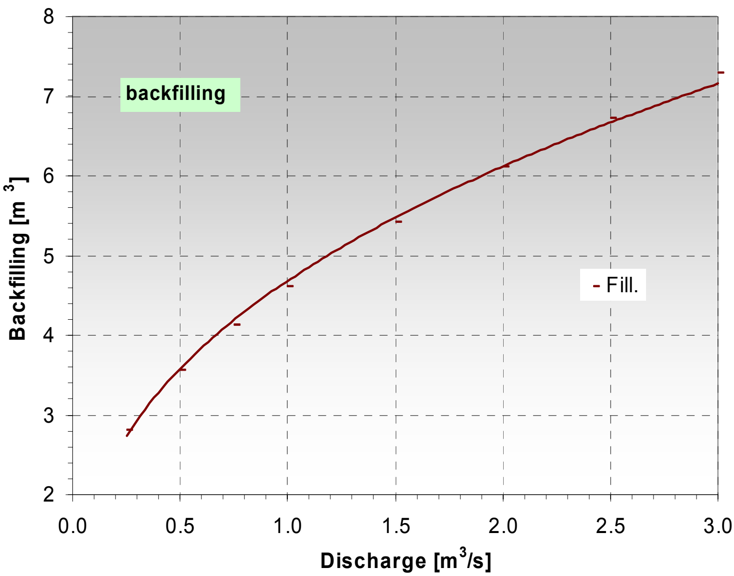
C2.4.4: Concrete volume, reinforcement, excavation, formwork and backfilling of buried pipe as a function of discharge and for S=0.002



For unit length of buried pipe

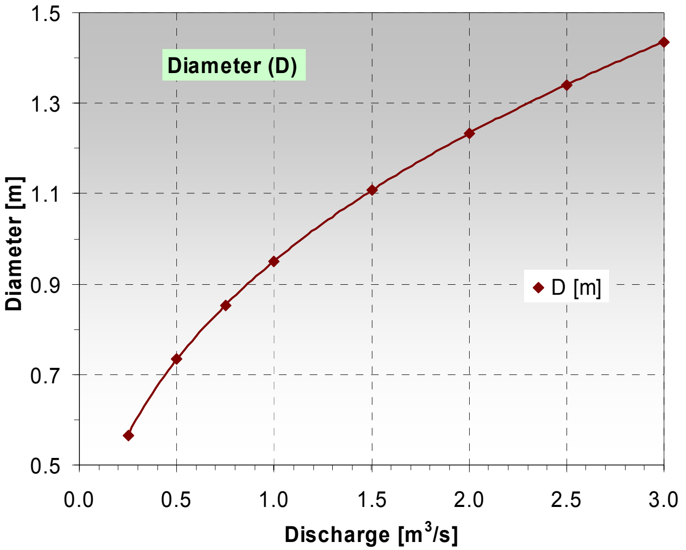
C2.4.4: Concrete volume, reinforcement, excavation, formwork and backfilling of buried pipe as a function of discharge and for S=0.002



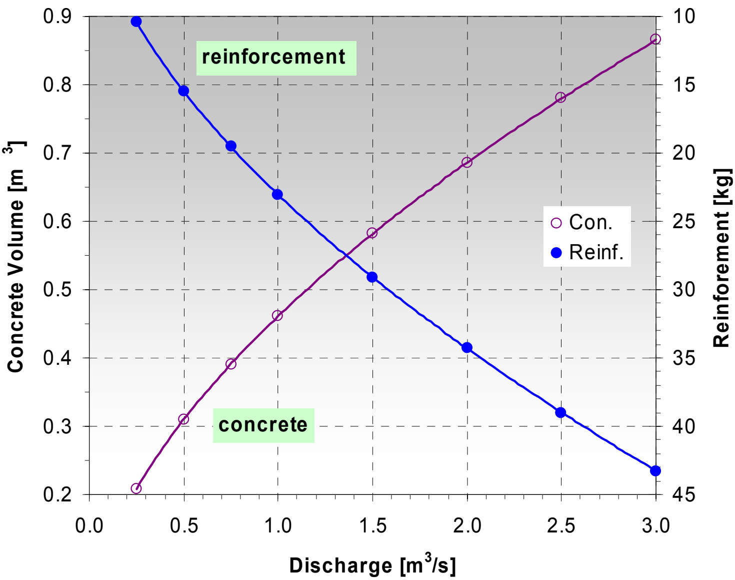


For unit length of buried pipe

C2.4.5: Diameter of buried pipe as a function of discharge and for S=0.003

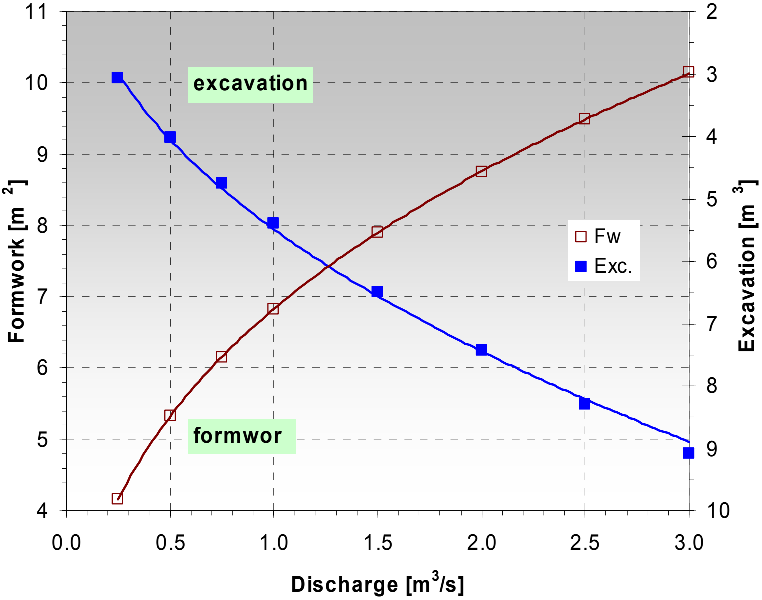


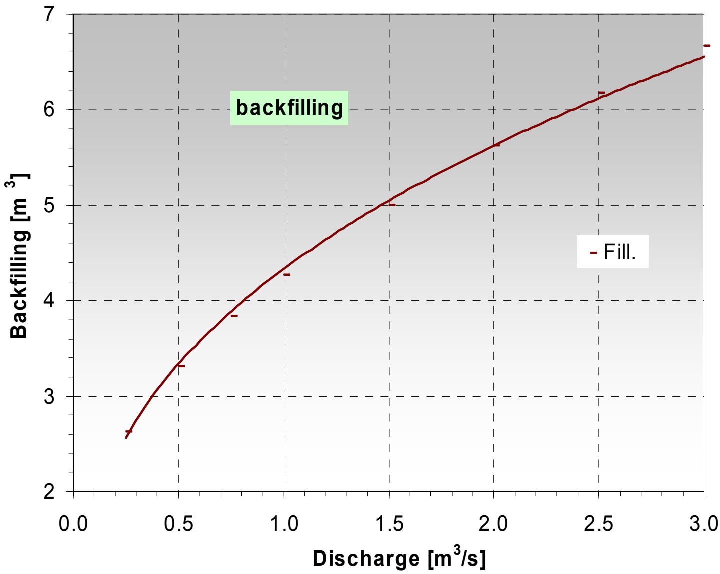
C2.4.6: Concrete volume, reinforcement, excavation, formwork and backfilling of buried pipe as a function of discharge and for S=0.003



For unit length of buried pipe

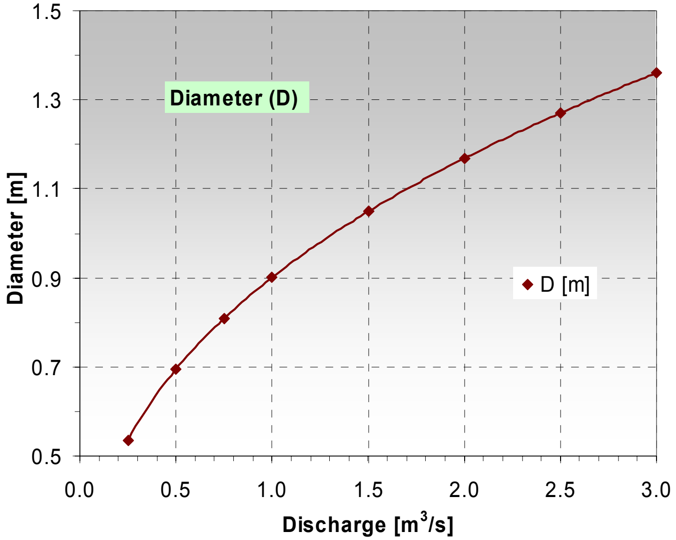
C2.4.6: Concrete volume, reinforcement, excavation, formwork and backfilling of buried pipe as a function of discharge and for S=0.003



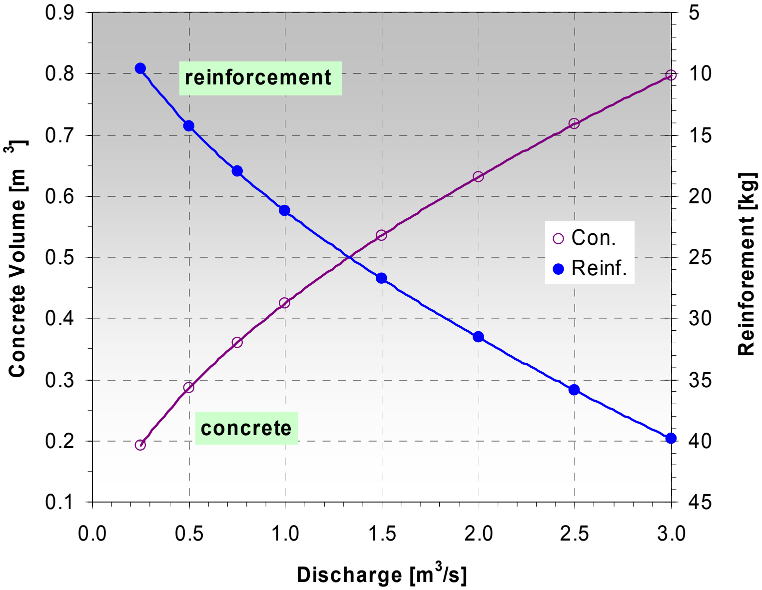


For unit length of buried pipe

C2.4.7: Diameter of buried pipe as a function of discharge and for S=0.004

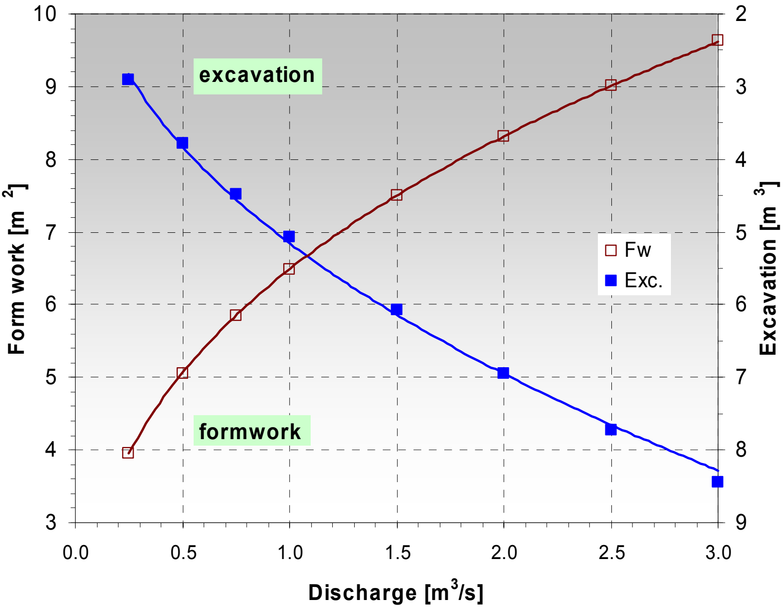


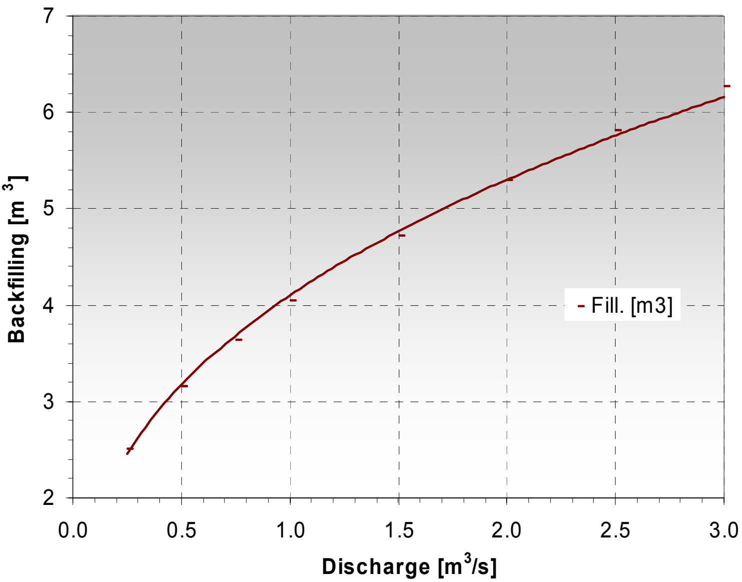
C2.4.8: Concrete volume, reinforcement, excavation, formwork and backfilling of buried pipe as a function of discharge and for S=0.004



For unit length of buried pipe

C2.4.8: Concrete volume, reinforcement, excavation, formwork and backfilling of buried pipe as a function of discharge and for S=0.004

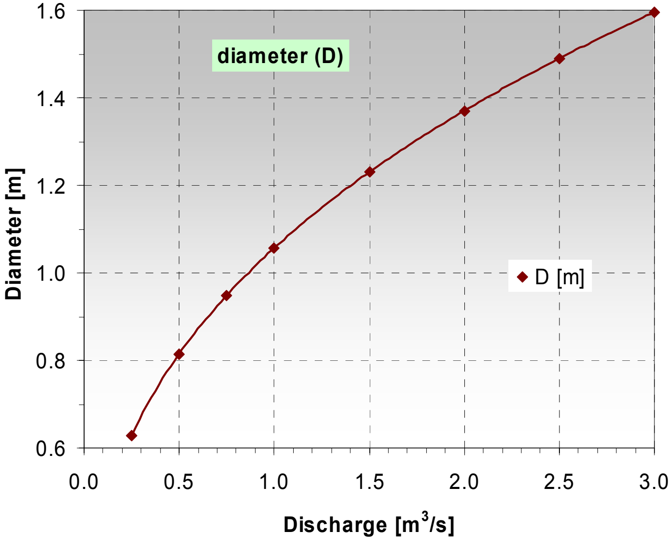




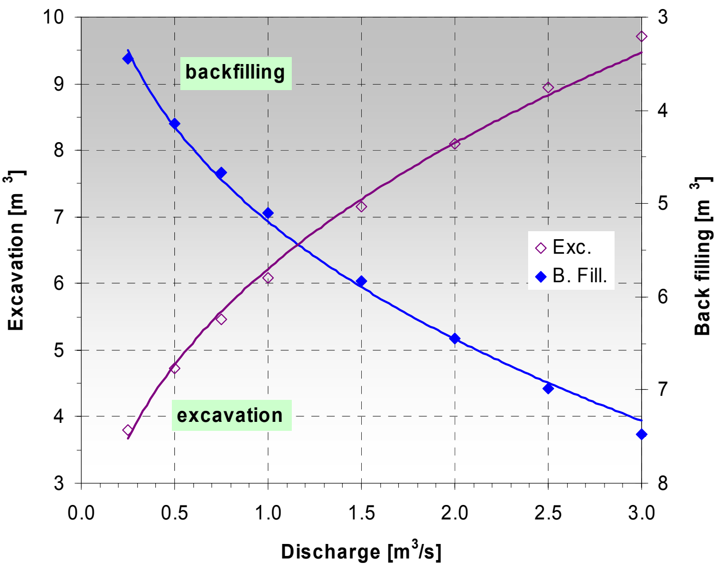
For unit length of buried pipe

C2.5: Buried PVC pipe

C2.5.1: Diameter of buried PVC pipe as a function of discharge and for S=0.001

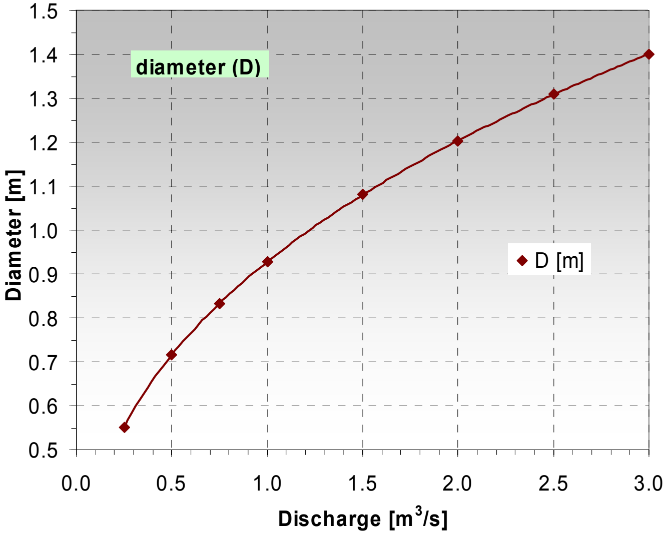


C2.5.2: Excavation and backfilling of buried PVC pipe as a function of discharge and for S=0.001

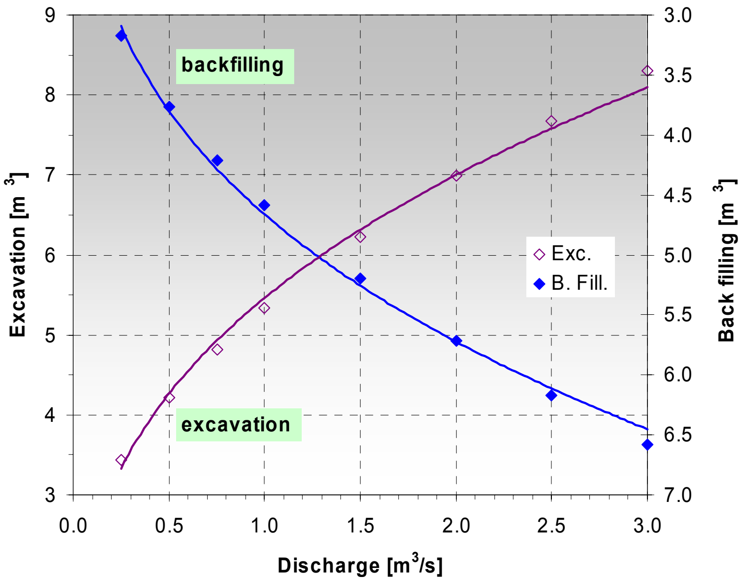


For unit length of buried PVC pipe

C2.5.3: Diameter of buried PVC pipe as a function of discharge and for S=0.002

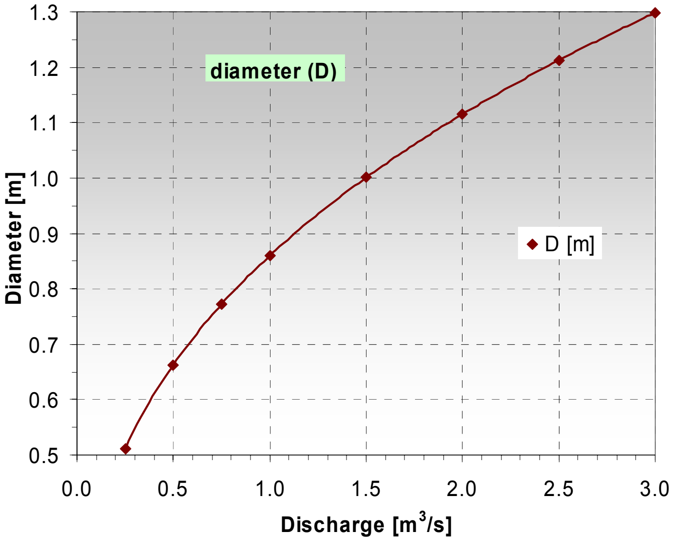


C2.5.4: Excavation and backfilling of buried PVC pipe as a function of discharge and for S=0.002

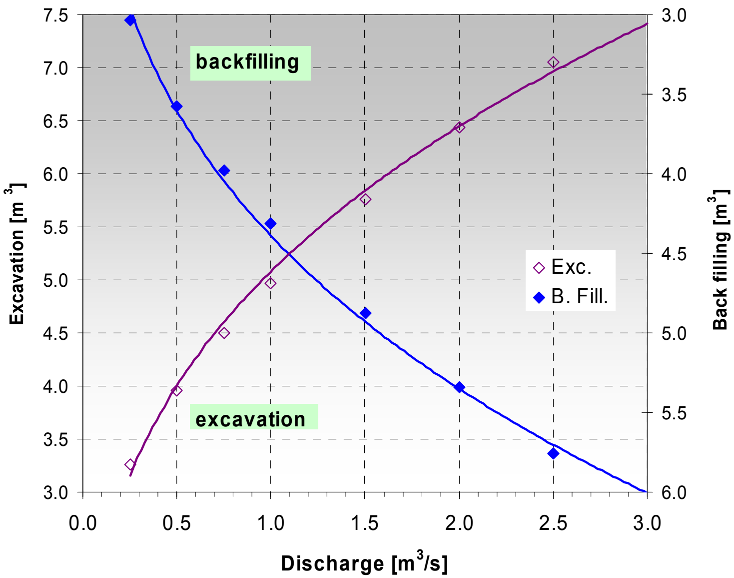


For unit length of buried PVC pipe

C2.5.5: Diameter of buried PVC pipe as a function of discharge and for S=0.003

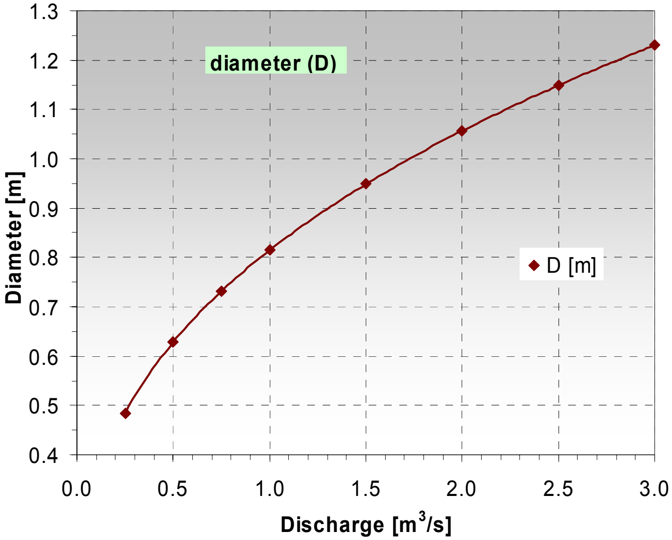


C2.5.6: Excavation and backfilling of buried PVC pipe as a function of discharge and for S=0.003

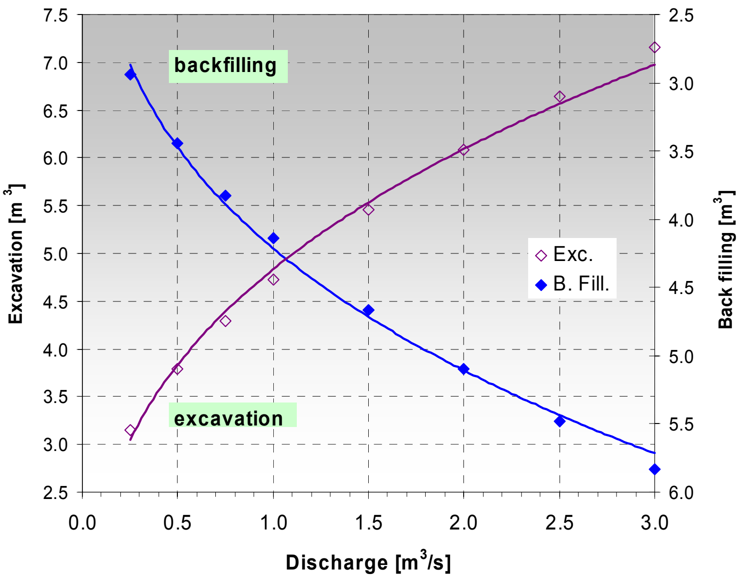


For unit length of buried PVC pipe

C2.5.7: Diameter of buried PVC pipe as a function of discharge and for S=0.004



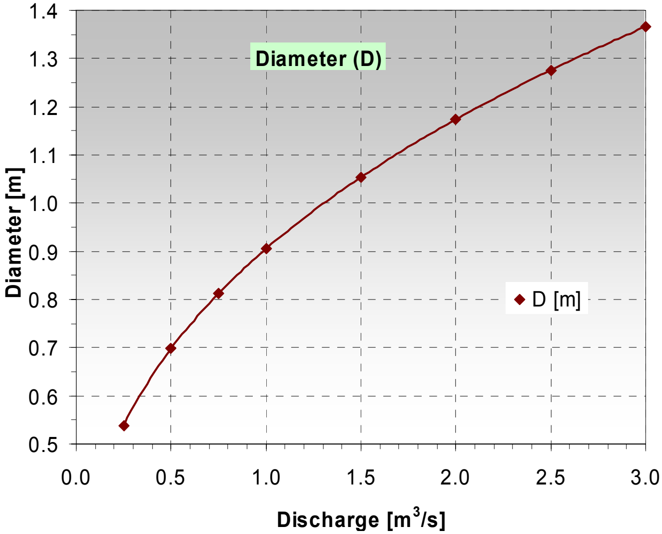
C2.5.8: Excavation and backfilling of buried PVC pipe as a function of discharge and for S=0.004



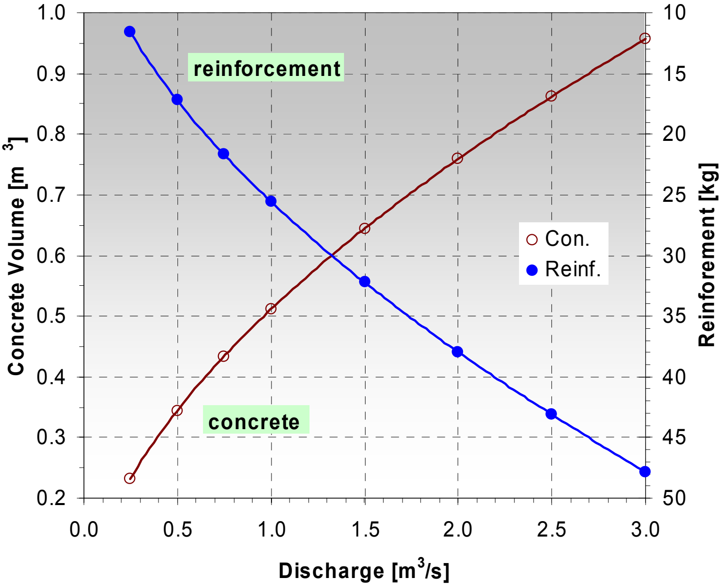
For unit length of buried PVC pipe

C2.6: Buried pipe (under pressure)

C2.6.1: Diameter of buried pipe (under pressure) as a function of discharge

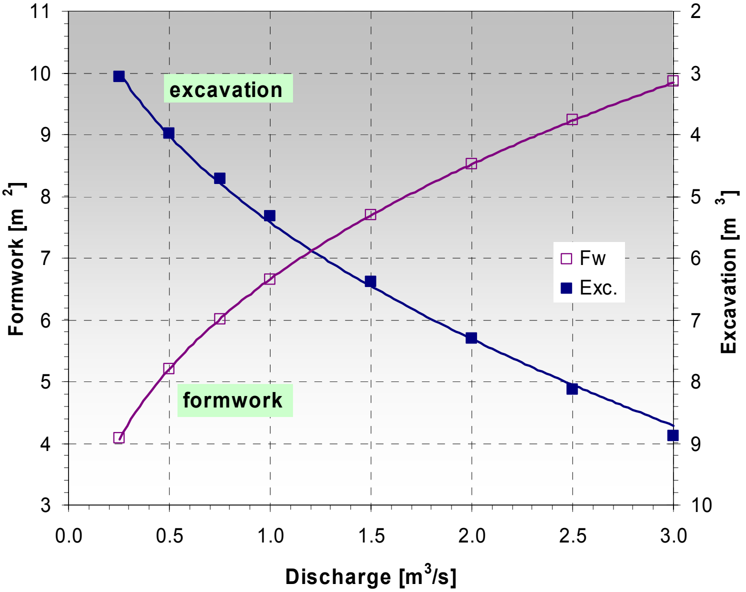


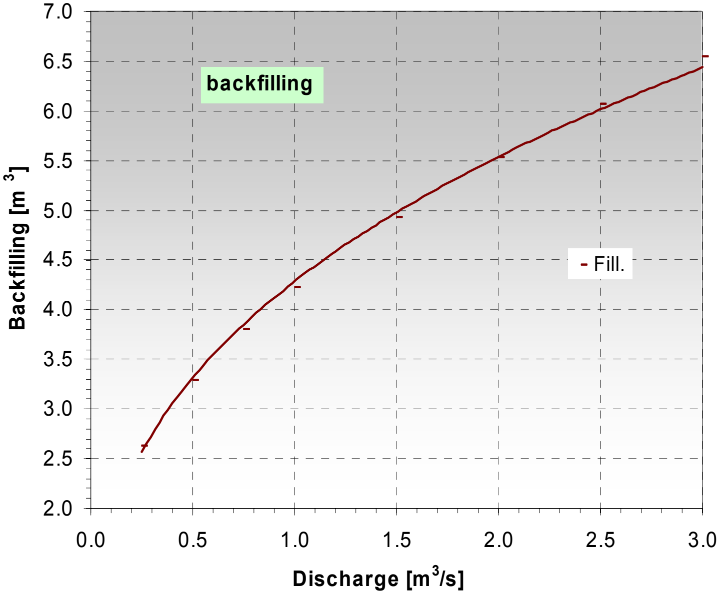
C2.6.2: Concrete volume, reinforcement, excavation, formwork and backfilling of buried pipe (under pressure) as a function of discharge



For unit length of buried pipe

C2.6.2: Concrete volume, reinforcement, excavation, formwork and backfilling of buried pipe (under pressure) as a function of discharge





For unit length of buried pipe

Appendix D

Standardization charts for forebay and surge tank

D1: Design tables of forebay

D1.1: Area and total depth of forebay as a function of discharge

Q: Design discharge (m^3/s)

H: Total height of forebay (m)

A: Area of forebay in the plan (m^2)

Area	discharge	width	lenght	spillway width
A	Q [m^3/s]	B [m]	L [m]	$B_{spillway}$ [m]
65	0.25	4.00	16.1	0.7
69	0.50	4.50	15.3	0.8
74	0.75	5.00	14.7	0.9
78	1.00	5.25	14.9	1.0
87	1.50	5.50	15.8	1.3
96	2.00	5.75	16.7	1.4
105	2.50	6.00	17.5	1.7
114	3.00	6.50	17.5	2.0

Values = c * Q + d

Values: Plan area [A]

Values	c	d
A	18	60

Values = c * Q ^{d}

Values: Total height [H]

Values	c	d
H	4.973	0.362

D1.2: Concrete volume, reinforcement, excavation and formwork of forebay as a function of discharge + forebay dimensions

Values = c * Q + d **Values: concrete volume reinforcment, excavation**

Rocky bed

concrete volume [m^3]

Component	c	d
Con.	71.3	16.1

reinforcement [kg]

Component	c	d
Reinf.	9526	1464

excavation [m^3]

Component	c	d
Exc.	451.0	281.0

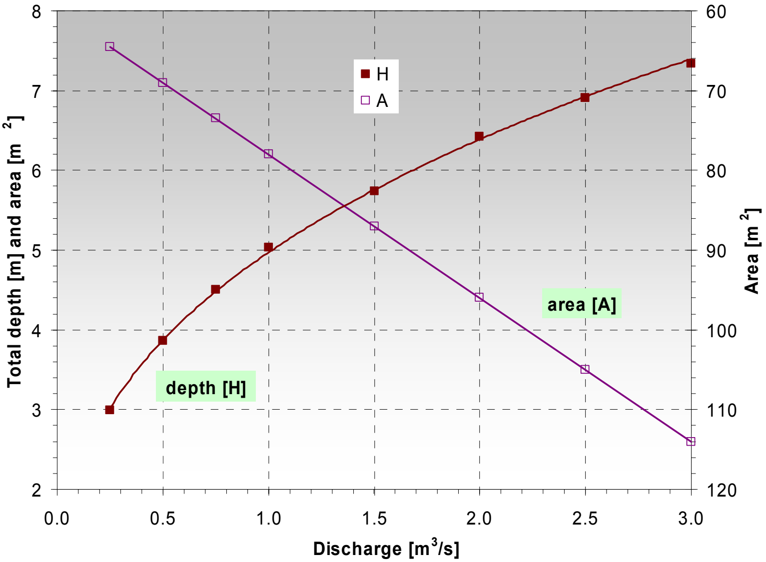
Values = c * Q^3 + d * Q^2 + e *Q + f **Values: formwork**

formwork [m^2]

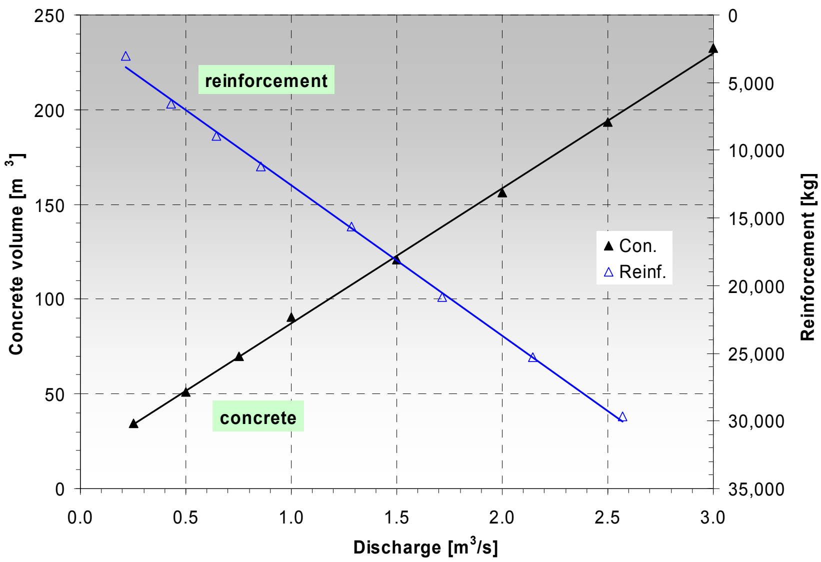
Component	c	d	e	f
Fw	1.3	-21.2	170.1	122.4

D2: Design charts of forebay

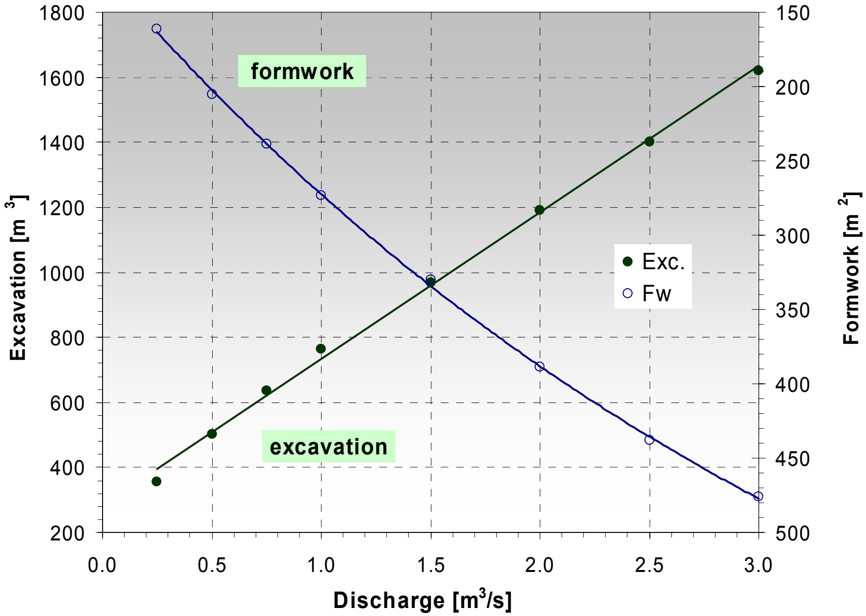
D2.1: Area and total depth of forebay as a function of discharge



D2.2: Concrete volume, reinforcement, excavation and formwork of forebay as a function of discharge



D2.2: Concrete volume, reinforcement, excavation and formwork of forebay as a function of discharge



D3: Design tables of surge tank

D3.1: Total depth of surge tank as a function of discharge

Q: design discharge (m^3/s)

H: Total height of surge tank (m)

L: Length of headrace pipe (under pressure) (m)

$$\text{Values} = c * L^3 + d * L^2 + e * L + f$$

Values: Total height [H]

Total height [m]

Q	c	d	e	f
0.25	3.00E-09	-8.00E-06	0.0090	3.08
0.50	4.00E-09	-1.00E-05	0.0110	3.76
0.75	5.00E-09	-1.00E-05	0.0128	4.32
1.00	5.00E-09	-1.00E-05	0.0140	4.78
1.50	6.00E-09	-1.00E-05	0.0147	5.44
2.00	6.00E-09	-1.00E-05	0.0153	5.89
2.50	6.00E-09	-1.00E-05	0.0163	6.43
3.00	7.00E-09	-2.00E-05	0.0171	6.96

D3.2: Concrete volume, formwork, excavation and reinforcement of surge tank as a function of discharge and length of headrace pipe

$$\text{Values} = c * L^3 + d * L^2 + e * L + f$$

Values: concrete volume and formwork

Rocky bed

Concrete volume [m^3]

Q	c	d	e	f
0.25	3.86E-09	-9.06E-06	0.010	4.7
0.50	8.51E-09	-2.00E-05	0.022	10.5
0.75	1.32E-08	-3.09E-05	0.034	16.2
1.00	1.84E-08	-4.31E-05	0.048	23.0
1.50	2.87E-08	-6.74E-05	0.075	39.8
2.00	3.44E-08	-8.07E-05	0.090	49.9
2.50	4.42E-08	-1.04E-04	0.116	65.8
3.00	5.21E-08	-1.22E-04	0.137	79.6

Formwork [m^2]

Q	c	d	e	f
0.25	2.21E-08	-5.18E-05	0.057	20.6
0.50	3.40E-08	-7.99E-05	0.088	32.1
0.75	4.38E-08	-1.03E-04	0.114	41.3
1.00	5.25E-08	-1.23E-04	0.137	50.2
1.50	6.76E-08	-1.59E-04	0.177	70.4
2.00	8.09E-08	-1.90E-04	0.212	87.6
2.50	9.30E-08	-2.18E-04	0.244	103.4
3.00	1.04E-07	-2.45E-04	0.274	119.3

D3.2: Concrete volume, formwork, excavation and reinforcement of surge tank as a function of discharge and length of headrace pipe

Values = c * L^3+ d * L^2 + e * L + f

Values: excavation and reinforcement

Rocky bed

Excavation [m^3]

Q	c	d	e	f
0.25	1.26E-08	-2.95E-05	0.0324	12.4
0.50	2.55E-08	-5.99E-05	0.0662	25.6
0.75	3.77E-08	-8.84E-05	0.0982	37.8
1.00	5.05E-08	-1.19E-04	0.1320	51.7
1.50	7.94E-08	-1.86E-04	0.2078	88.8
2.00	1.04E-07	-2.45E-04	0.2735	120.6
2.50	1.31E-07	-3.08E-04	0.3441	155.9
3.00	1.56E-07	-3.67E-04	0.4105	190.9

Values = c * L^2+ d * L + e

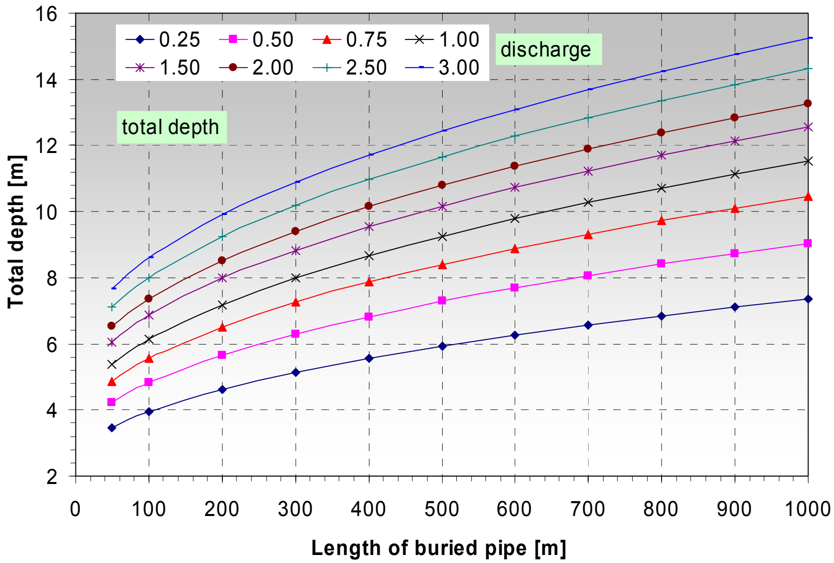
Values: reinforcement

Reinforcement [kg]

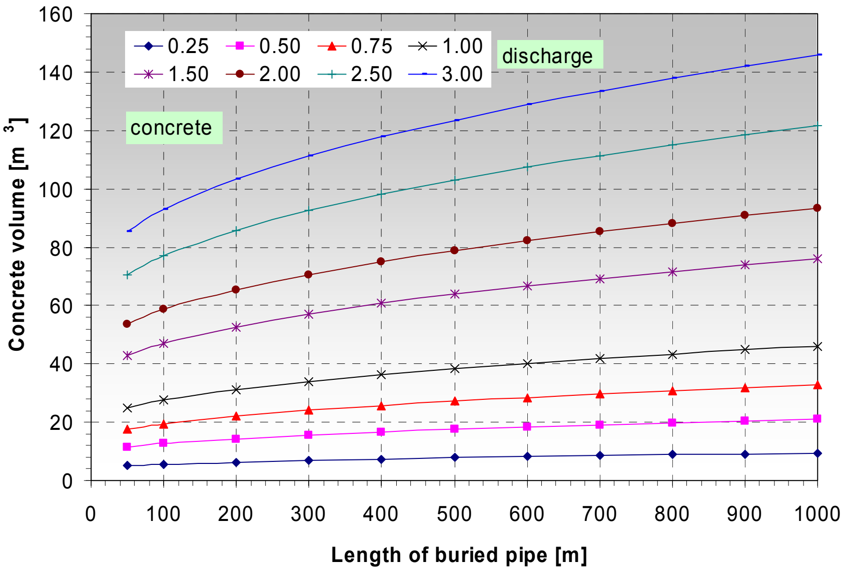
Q	c	d	e
0.25	6.30E-04	1.86	372
0.50	1.47E-03	3.03	898
0.75	2.50E-03	4.62	1383
1.00	3.52E-03	5.97	1992
1.50	4.94E-03	8.03	3574
2.00	5.89E-03	12.95	4289
2.50	7.48E-03	16.58	5671
3.00	8.67E-03	22.40	6738

D4: Design charts of surge tank

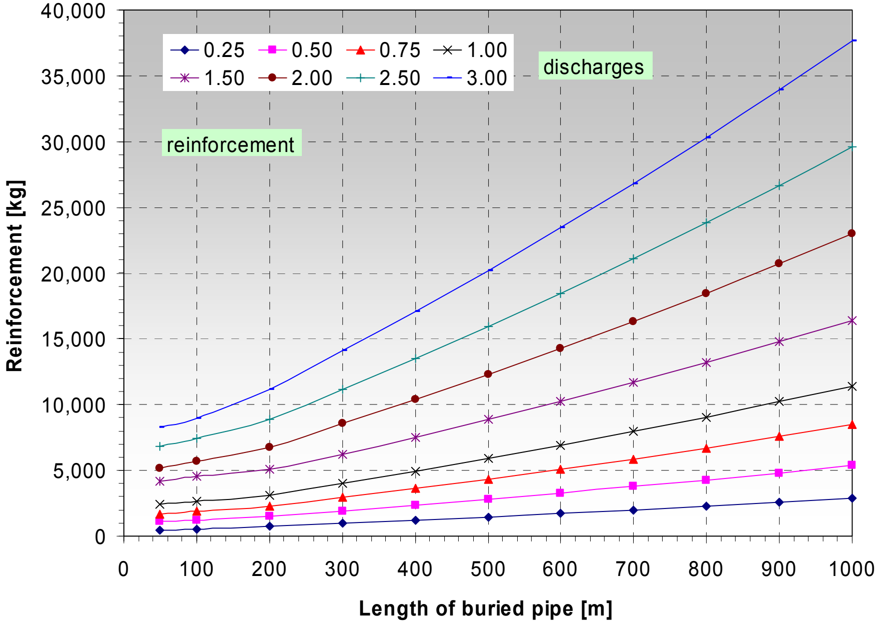
D4.1: Total depth of surge tank as a function of discharge and headrace pipe length

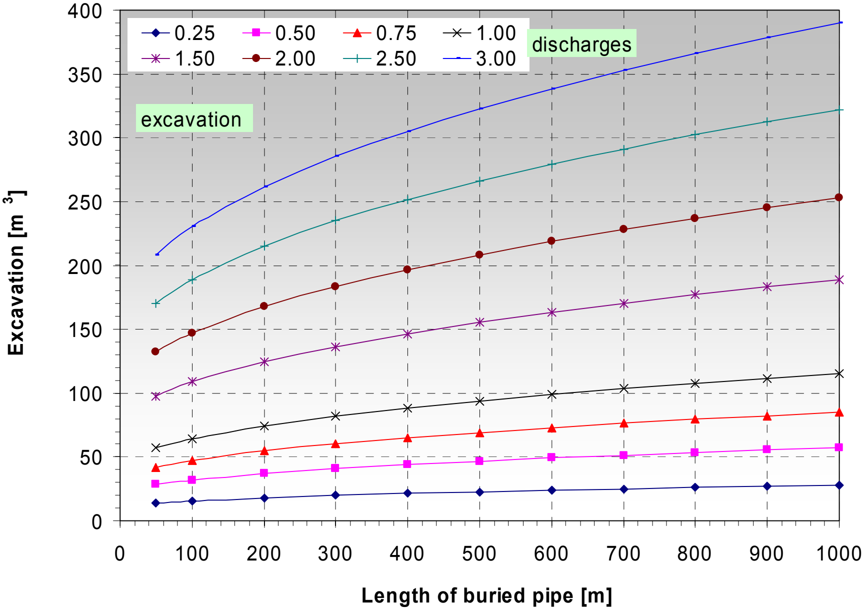


D4.2: Concrete volume, reinforcement, excavation and formwork of surge tank as a function of discharge and length of headrace pipe (under pressure)

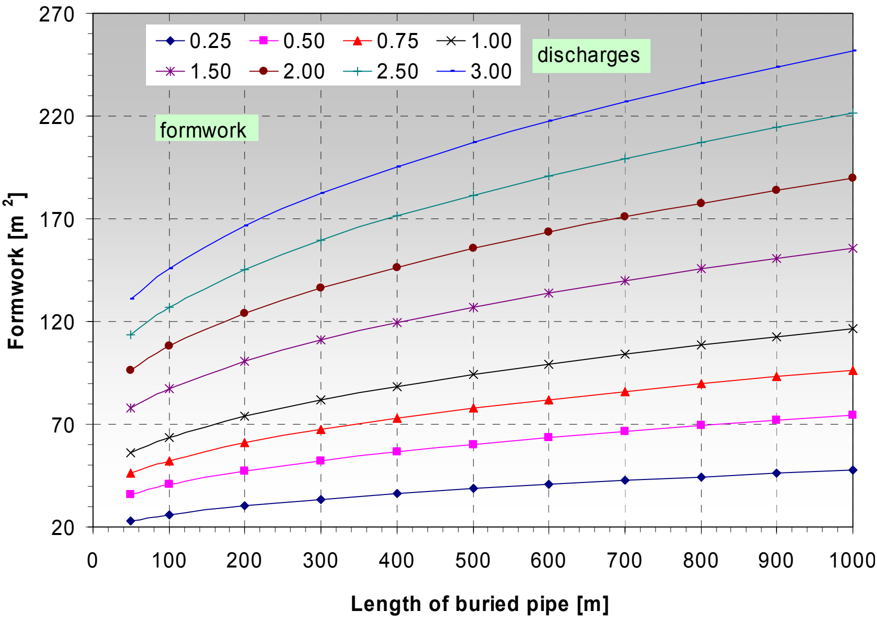


D4.2: Concrete volume, reinforcement, excavation and formwork of surge tank as a function of discharge and length of headrace pipe (under pressure)





D4.2: Concrete volume, reinforcement, excavation and formwork of surge tank as a function of discharge and length of headrace pipe (under pressure)



Appendix E

Standardization charts for penstock

E1: Design tables of penstock

E1.1: Optimum penstock diameter, wall thickness and velocity as a function of discharge, energy sale price and total head

D: Optimum penstock diameter (m)
H: Total head (m)

Q: design discharge (m^3/s)
E.S.P. : energy sale price (CHF/kWh)

Values = $c * Q^d$ **Value: Optimum diameter, velocity and thickness of penstock**

Diameter (m) for H=100m

E.S.P.	c	d
0.04	0.800	0.450
0.08	0.871	0.446
0.12	0.919	0.443
0.16	0.953	0.442

Thickness (mm) for H=100m

E.S.P.	c	d
0.04	4.561	0.529
0.08	4.366	0.524
0.12	4.274	0.520
0.16	4.224	0.517

Velocity (m/s) for H=100m

E.S.P.	c	d
0.04	1.402	0.117
0.08	1.509	0.115
0.12	1.679	0.108
0.16	1.991	0.100

Diameter (m) for H=200m

E.S.P.	c	d
0.04	0.723	0.433
0.08	0.793	0.429
0.12	0.838	0.428
0.16	0.870	0.428

Thickness (mm) for H=200m

E.S.P.	c	d
0.04	6.529	0.500
0.08	6.539	0.504
0.12	6.576	0.510
0.16	6.706	0.518

Velocity (m/s) for H=200m

E.S.P.	c	d
0.04	1.683	0.144
0.08	1.815	0.144
0.12	2.023	0.141
0.16	2.436	0.135

Diameter (m) for H=300m

E.S.P.	c	d
0.04	0.683	0.427
0.08	0.751	0.424
0.12	0.794	0.425
0.16	0.824	0.423

Thickness (mm) for H=300m

E.S.P.	c	d
0.04	8.467	0.508
0.08	8.432	0.498
0.12	8.455	0.493
0.16	8.491	0.489

Velocity (m/s) for H=300m

E.S.P.	c	d
0.04	1.875	0.155
0.08	2.022	0.151
0.12	2.256	0.152
0.16	2.726	0.147

Diameter (m) for H=400m

E.S.P.	c	d
0.04	0.658	0.423
0.08	0.722	0.421
0.12	0.764	0.421
0.16	0.794	0.422

Thickness (mm) for H=400m

E.S.P.	c	d
0.04	10.042	0.499
0.08	10.104	0.490
0.12	10.191	0.484
0.16	10.274	0.481

Velocity (m/s) for H=400m

E.S.P.	c	d
0.04	2.020	0.157
0.08	2.184	0.159
0.12	2.441	0.158
0.16	2.944	0.155

Diameter (m) for H=500m

E.S.P.	c	d
0.04	0.637	0.421
0.08	0.700	0.419
0.12	0.740	0.421
0.16	0.770	0.420

Thickness (mm) for H=500m

E.S.P.	c	d
0.04	11.498	0.492
0.08	11.659	0.483
0.12	11.808	0.479
0.16	11.938	0.475

Velocity (m/s) for H=500m

E.S.P.	c	d
0.04	2.150	0.161
0.08	2.324	0.158
0.12	2.597	0.161
0.16	3.142	0.158

If $t_{cal}<t_{min}$ then $t_{design}=t_{min}$ (for example: t_{min}=4mm)

E1.1: Optimum penstock diameter, wall thickness and velocity as a function of discharge, energy sale price and total head

Values = c * Q^d **Value: Optimum diameter, velocity and thickness of penstock**

Diameter (m) for H=600m

E.S.P.	c	d
0.04	0.622	0.415
0.08	0.683	0.420
0.12	0.722	0.419
0.16	0.751	0.417

Thickness (mm) for H=600m

E.S.P.	c	d
0.04	12.880	0.486
0.08	13.129	0.479
0.12	13.342	0.474
0.16	13.526	0.470

Velocity (m/s) for H=600m

E.S.P.	c	d
0.04	2.256	0.166
0.08	2.445	0.162
0.12	2.733	0.160
0.16	3.294	0.171

Diameter (m) for H=700m

E.S.P.	c	d
0.04	0.609	0.417
0.08	0.670	0.415
0.12	0.707	0.418
0.16	0.736	0.417

Thickness (mm) for H=700m

E.S.P.	c	d
0.04	14.194	0.482
0.08	14.555	0.473
0.12	14.819	0.469
0.16	15.052	0.466

Velocity (m/s) for H=700m

E.S.P.	c	d
0.04	2.348	0.166
0.08	2.547	0.165
0.12	2.835	0.170
0.16	3.433	0.167

Diameter (m) for H=800m

E.S.P.	c	d
0.04	0.598	0.417
0.08	0.657	0.416
0.12	0.695	0.416
0.16	0.721	0.418

Thickness (mm) for H=800m

E.S.P.	c	d
0.04	15.459	0.478
0.08	15.910	0.470
0.12	16.256	0.465
0.16	16.503	0.464

Velocity (m/s) for H=800m

E.S.P.	c	d
0.04	2.449	0.164
0.08	2.633	0.169
0.12	2.950	0.167
0.16	3.563	0.165

Diameter (m) for H=900m

E.S.P.	c	d
0.04	0.588	0.416
0.08	0.646	0.417
0.12	0.683	0.415
0.16	0.710	0.419

Thickness (mm) for H=900m

E.S.P.	c	d
0.04	16.684	0.474
0.08	17.226	0.468
0.12	17.632	0.462
0.16	17.938	0.461

Velocity (m/s) for H=900m

E.S.P.	c	d
0.04	2.525	0.163
0.08	2.727	0.171
0.12	3.052	0.165
0.16	3.685	0.168

Diameter (m) for H=1000m

E.S.P.	c	d
0.04	0.580	0.422
0.08	0.638	0.418
0.12	0.674	0.420
0.16	0.703	0.416

Thickness (mm) for H=1000m

E.S.P.	c	d
0.04	17.871	0.473
0.08	18.517	0.464
0.12	18.974	0.461
0.16	19.363	0.456

Velocity (m/s) for H=1000m

E.S.P.	c	d
0.04	2.576	0.168
0.08	2.803	0.161
0.12	3.131	0.163
0.16	3.787	0.155

If $t_{cal}<t_{min}$ then $t_{design}=t_{min}$ (for example: t_{min}=4mm)

E1.2: Concrete volume, reinforcement, excavation and formwork of anchor block as a function of deflection angle, discharge and total head for E.S.P. =0.16 CHF/kWh

Q: design discharge (m^3/s)

Energy sale price=0.16 CHF/kWh

Angle: Deflection angle of penstock in position of anchor block (^{o})

H: Total head (m)

Values = c * Q^d

Value: Concrete volume (Con.), reinforcement (Reinf.), excavation (Exc.), formwork (Fw)

Con. (m^3) , H=100m

Angle	c	d
120	151	0.9561
140	112	0.9561
160	59	0.9561

Reinf. (kg) , H=100m

Angle	c	d
120	7980	0.9561
140	5895	0.9561
160	3121	0.9561

Exc. (m^3) , H=100m

Angle	c	d
120	91	0.9561
140	67	0.9561
160	36	0.9561

Fw (m^2) , H=100m

Angle	c	d
120	130	0.6471
140	104	0.6414
160	66	0.6304

Con. (m^3) , H=200m

Angle	c	d
120	219	0.8921
140	160	0.9244
160	85	0.9244

Reinf. (kg) , H=200m

Angle	c	d
120	11576	0.8921
140	8446	0.9244
160	4489	0.9244

Exc. (m^3) , H=200m

Angle	c	d
120	132	0.8921
140	96	0.9244
160	51	0.9244

Fw (m^2) , H=200m

Angle	c	d
120	171	0.6080
140	135	0.6251
160	85	x0.6135

Con. (m^3) , H=300m

Angle	c	d
120	266	0.9041
140	196	0.9154
160	104	0.9154

Reinf. (kg) , H=300m

Angle	c	d
120	14014	0.9041
140	10333	0.9154
160	5492	0.9154

Exc. (m^3) , H=300m

Angle	c	d
120	159	0.9041
140	118	0.9154
160	62	0.9154

Fw (m^2) , H=300m

Angle	c	d
120	197	0.6204
140	157	0.6224
160	99	0.6105

Con. (m^3) , H=400m

Angle	c	d
120	310	0.9053
140	230	0.8980
160	122	0.8980

Reinf. (kg) , H=400m

Angle	c	d
120	16337	0.9053
140	12117	0.8980
160	6440	0.8980

Exc. (m^3) , H=400m

Angle	c	d
120	186	0.9053
140	138	0.8980
160	73	0.8980

Fw (m^2) , H=400m

Angle	c	d
120	221	0.6245
140	177	0.6128
160	111	0.6007

Con. (m^3) , H=500m

Angle	c	d
120	350	0.8892
140	258	0.9005
160	137	0.9005

Reinf. (kg) , H=500m

Angle	c	d
120	18470	0.8892
140	13619	0.9005
160	7238	0.9005

Exc. (m^3) , H=500m

Angle	c	d
120	210	0.8892
140	155	0.9005
160	82	0.9005

Fw (m^2) , H=500m

Angle	c	d
120	242	0.6152
140	193	0.6171
160	121	0.6047

E1.2: Concrete volume, reinforcement, excavation and formwork of anchor block as a function of deflection angle, discharge and total head for E.S.P. =0.16 CHF/kWh

Q: design discharge (m^3/s)

Energy sale price=0.16 CHF/kWh

Angle: Deflection angle of penstock in position of anchor block (^{o})

H: Total head (m)

Values = c * Q^d

Value: Concrete volume (Con.), reinforcement (Reinf.), excavation (Exc.), formwork (Fw)

Con. (m^3) , H=600m

Angle	c	d
120	386	0.8842
140	286	0.8859
160	152	0.8859

Reinf. (kg) , H=600m

Angle	c	d
120	20339	0.8842
140	15073	0.8859
160	8010	0.8859

Exc. (m^3) , H=600m

Angle	c	d
120	231	0.8842
140	171	0.8859
160	91	0.8859

Fw (m^2) , H=600m

Angle	c	d
120	260	0.6137
140	208	0.6083
160	130	0.5959

Con. (m^3) , H=700m

Angle	c	d
120	420	0.8808
140	314	0.8838
160	167	0.8838

Reinf. (kg) , H=700m

Angle	c	d
120	22151	0.8808
140	16550	0.8838
160	8795	0.8838

Exc. (m^3) , H=700m

Angle	c	d
120	252	0.8808
140	188	0.8838
160	100	0.8838

Fw (m^2) , H=700m

Angle	c	d
120	277	0.6130
140	223	0.6087
160	139	0.5961

Con. (m^3) , H=800m

Angle	c	d
120	452	0.8810
140	332	0.8838
160	177	0.8838

Reinf. (kg) , H=800m

Angle	c	d
120	23853	0.8810
140	17529	0.8838
160	9316	0.8838

Exc. (m^3) , H=800m

Angle	c	d
120	271	0.8810
140	199	0.8838
160	106	0.8838

Fw (m^2) , H=800m

Angle	c	d
120	293	0.6146
140	232	0.6100
160	145	0.5972

Con. (m^3) , H=900m

Angle	c	d
120	483	0.8835
140	358	0.8837
160	190	0.8837

Reinf. (kg) , H=900m

Angle	c	d
120	25493	0.8835
140	18894	0.8837
160	10041	0.8837

Exc. (m^3) , H=900m

Angle	c	d
120	290	0.8835
140	215	0.8837
160	114	0.8837

Fw (m^2) ,H=900m

Angle	c	d
120	308	0.6180
140	246	0.6116
160	154	0.5986

Con. (m^3) , H=1000m

Angle	c	d
120	518	0.8855
140	382	0.8648
160	203	0.8648

Reinf. (kg) , H=1000m

Angle	c	d
120	27347	0.8855
140	20140	0.8648
160	10703	0.8648

Exc. (m^3) , H=1000m

Angle	c	d
120	311	0.8855
140	229	0.8648
160	122	0.8648

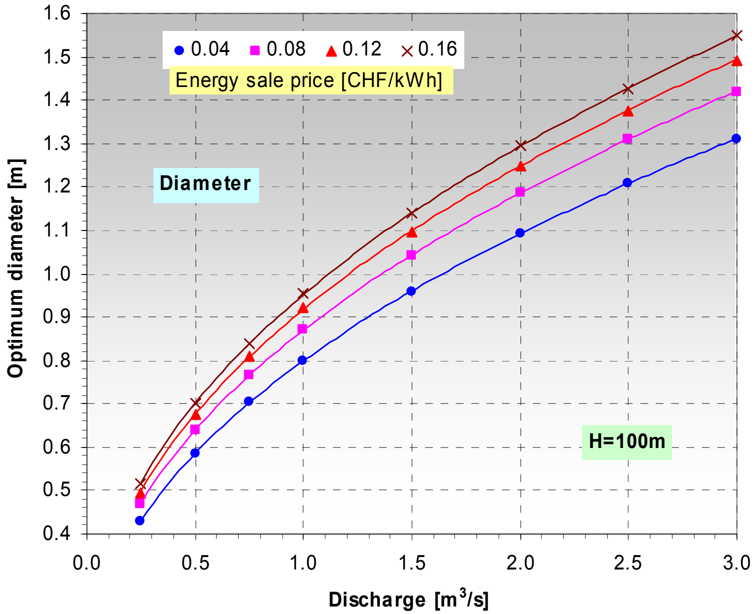
Fw (m^2) , H=1000m

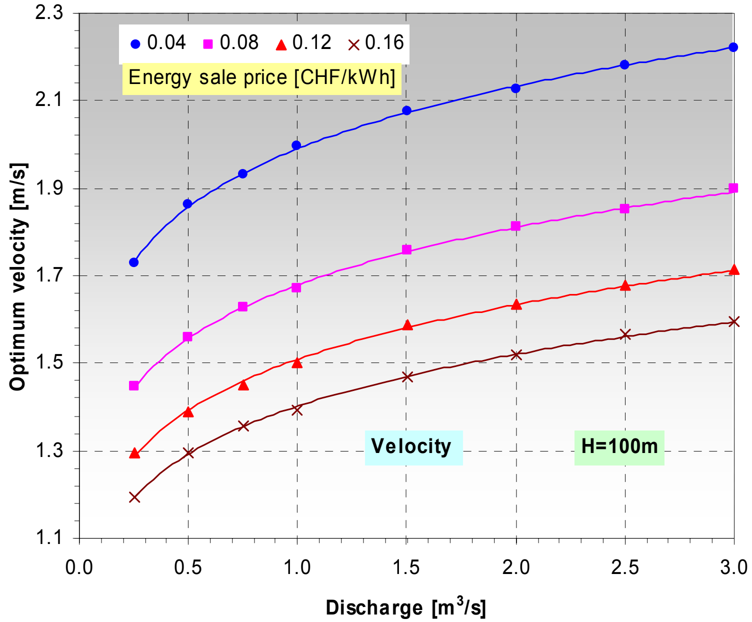
Angle	c	d
120	325	0.6214
140	258	0.5989
160	161	0.5860

E2: Design charts of penstock

E2.1: Optimum penstock diameter and velocity as a function of discharge and energy sale price for different total heads

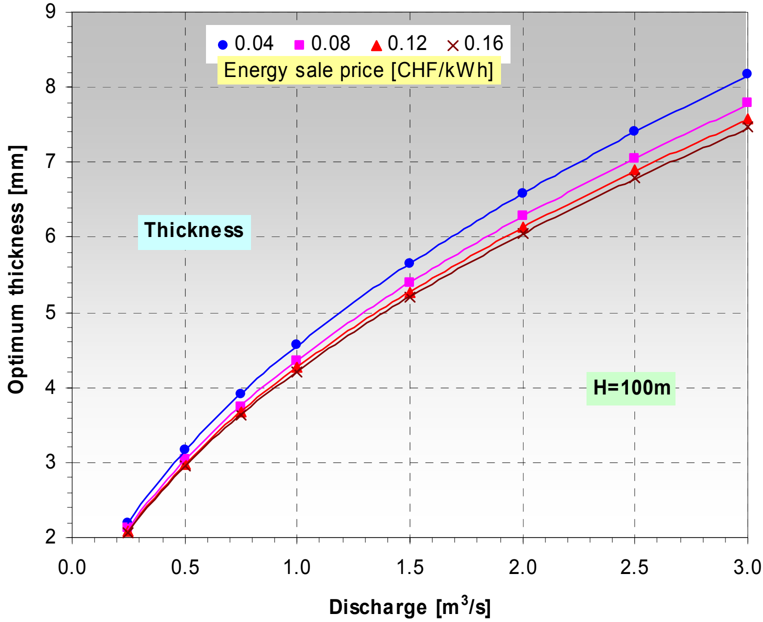
E2.1.1: H=100m



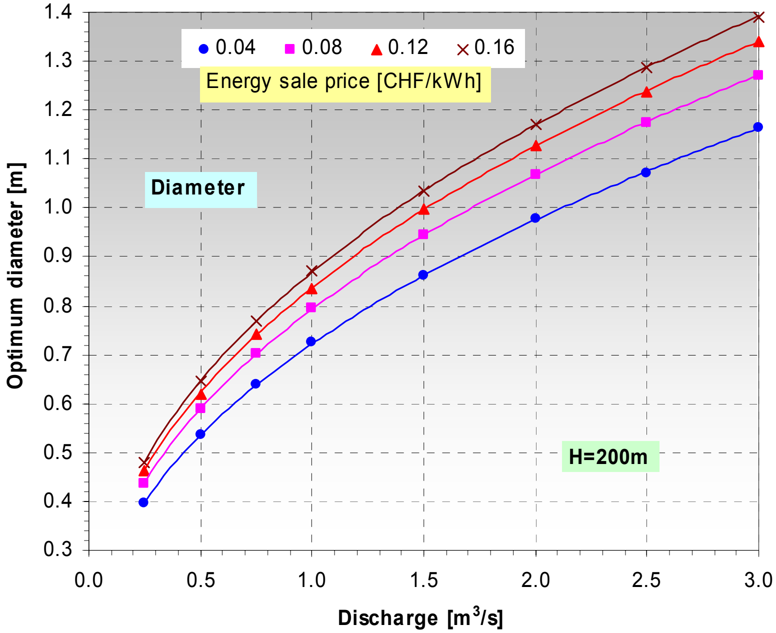


E2.1.1: H=100m

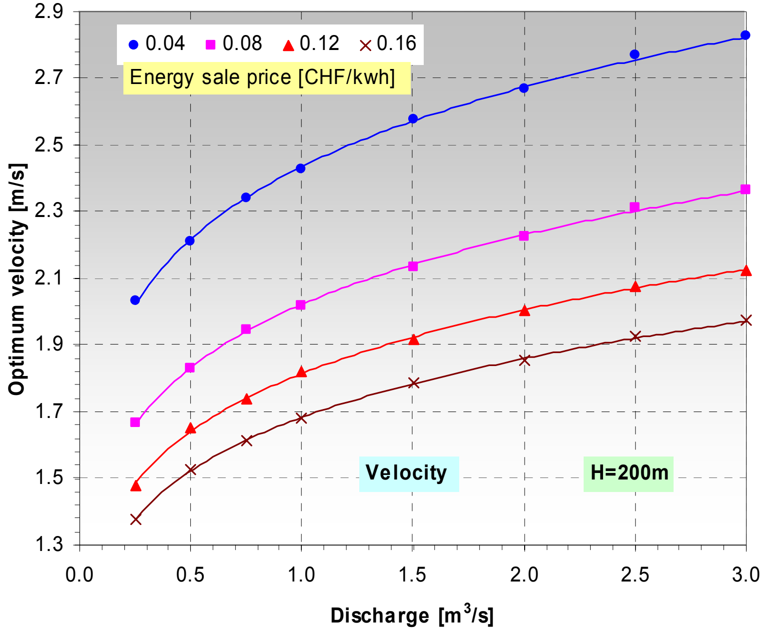
If $t_{graph} < t_{min}$ then $t_{design} = t_{min}$ (for example: t_{min}=4mm)



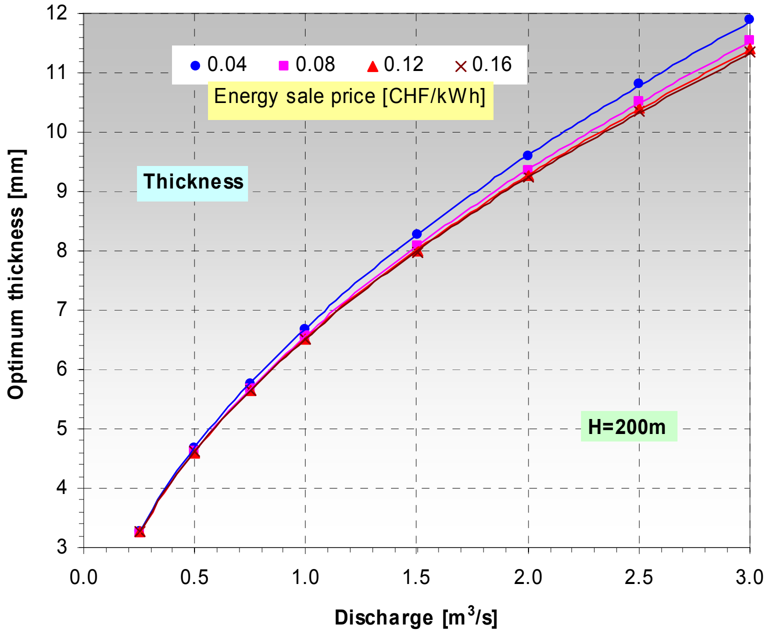
E2.1.2: H=200m



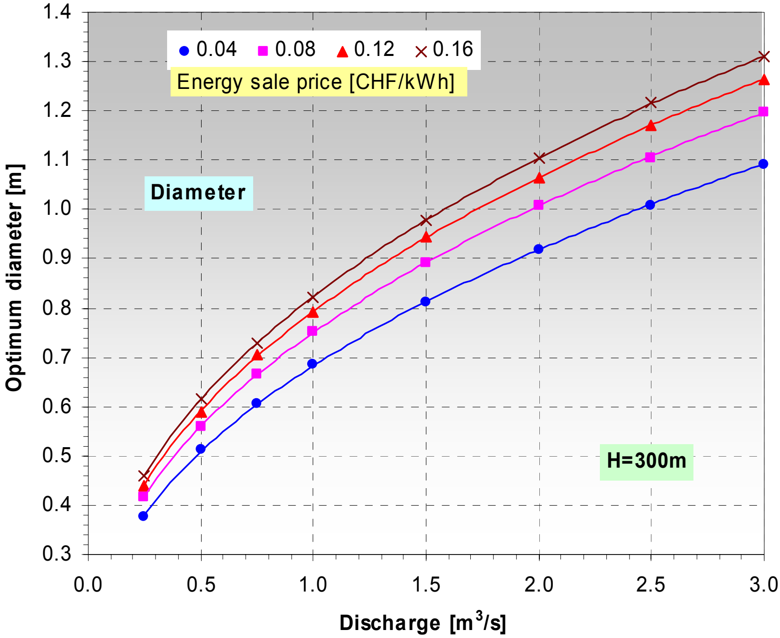
E2.1.2: H=200m



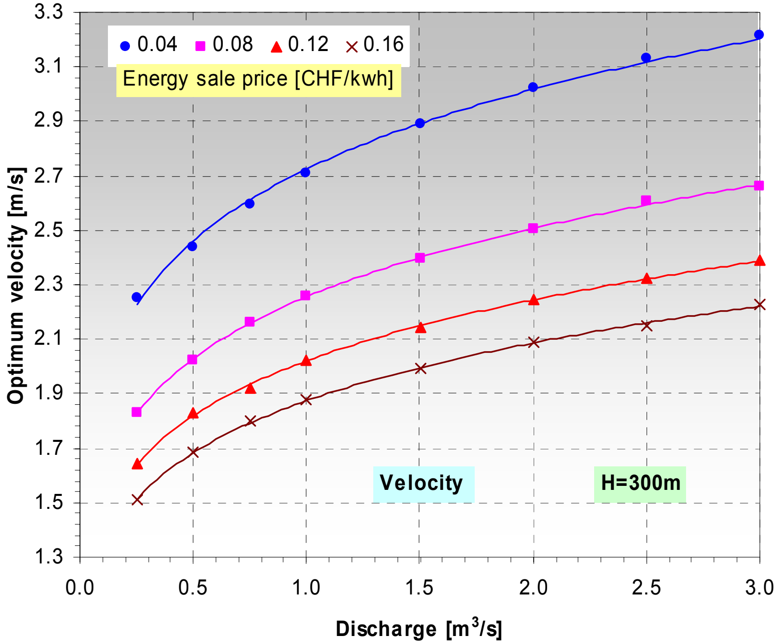
If $t_{graph}<t_{min}$ then $t_{design}=t_{min}$ (for example: t_{min}=4mm)



E2.1.3: H=300m



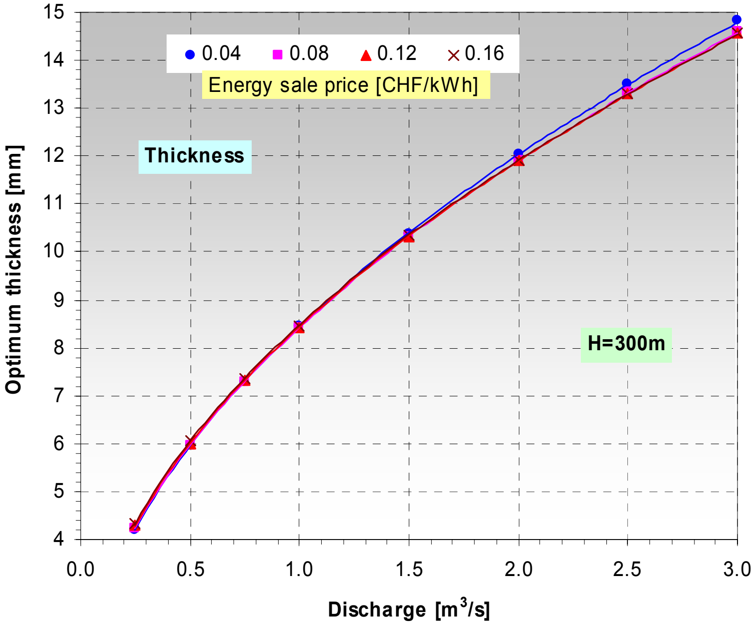
0.04
0.08
0.12
0.16
Energy sale price [CHF/kWh]
Diameter
H=300m
Optimum diameter [m]
Discharge [m3/s]
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.0
0.5
1.0
1.5
2.0
2.5
3.0



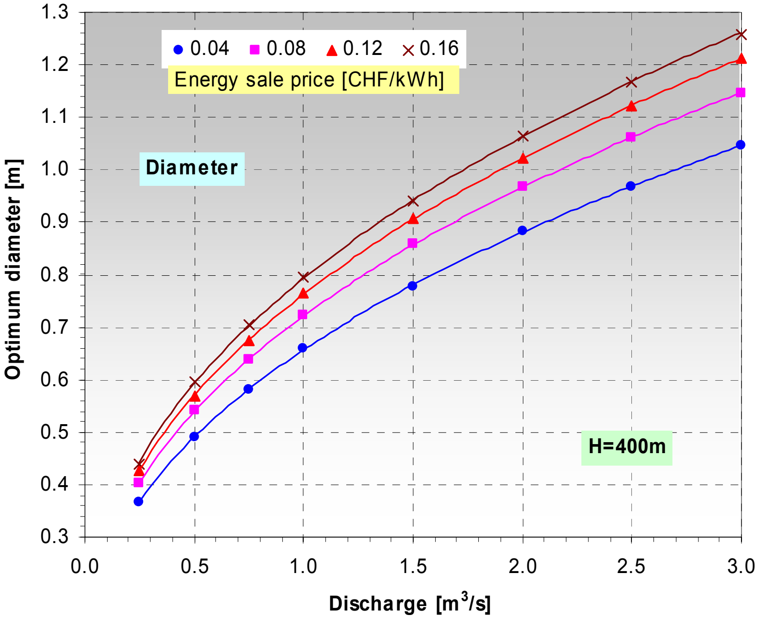
0.04
0.08
0.12
0.16
Energy sale price [CHF/kwh]
Velocity
H=300m
Optimum velocity [m/s]
Discharge [m3/s]
3.3
3.1
2.9
2.7
2.5
2.3
2.1
1.9
1.7
1.5
1.3
0.0
0.5
1.0
1.5
2.0
2.5
3.0

E2.1.3: H=300m

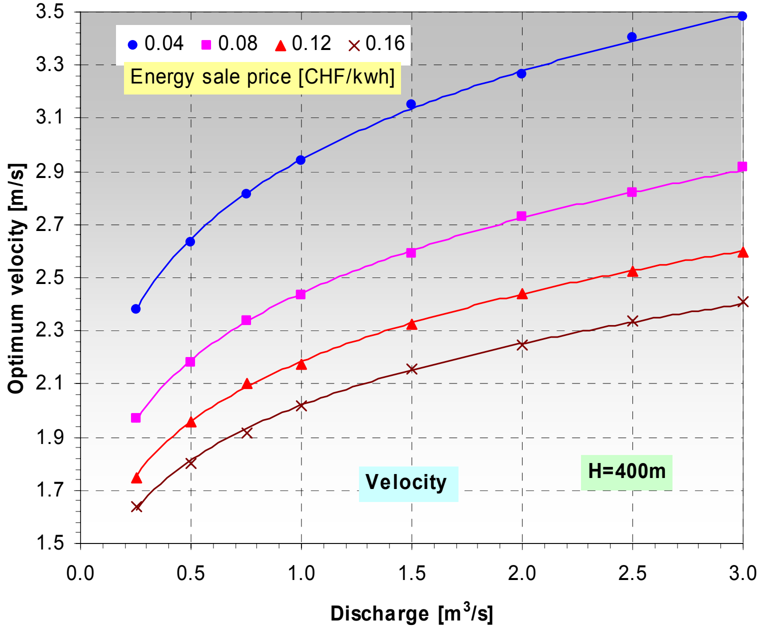
If $t_{graph}<t_{min}$ then $t_{design}=t_{min}$ (for example: t_{min}=4mm)



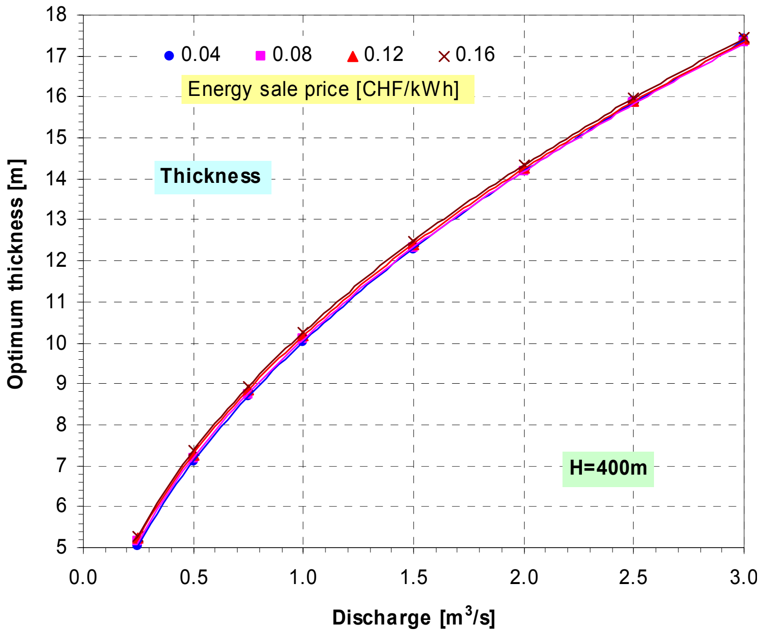
E2.1.4: H=400m



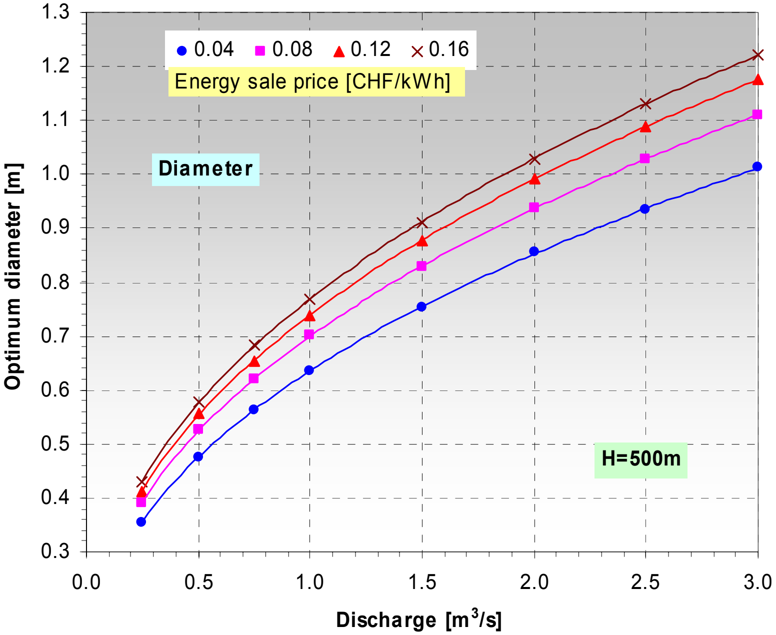
E2.1.4: H=400m



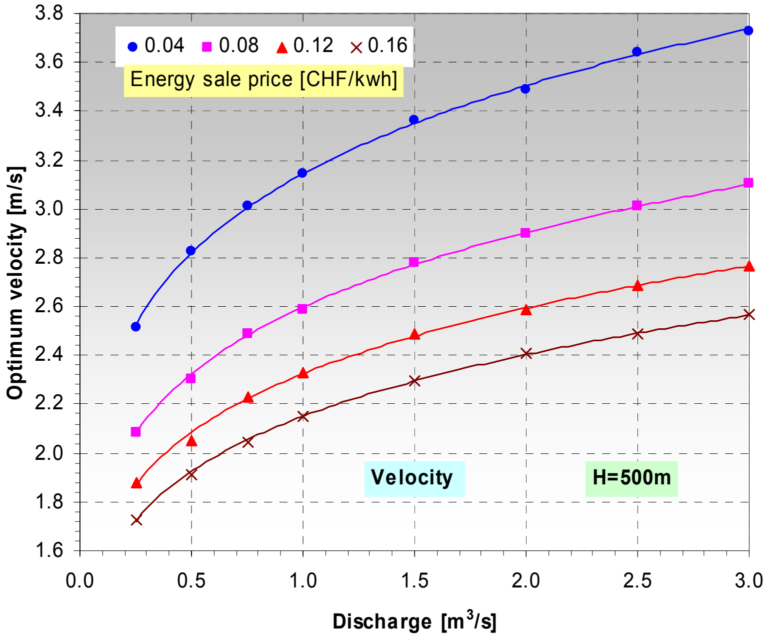
If t_{graph}<t_{min} then t_{design}=t_{min} (for example: t_{min}=4mm)



E2.1.5: H=500m

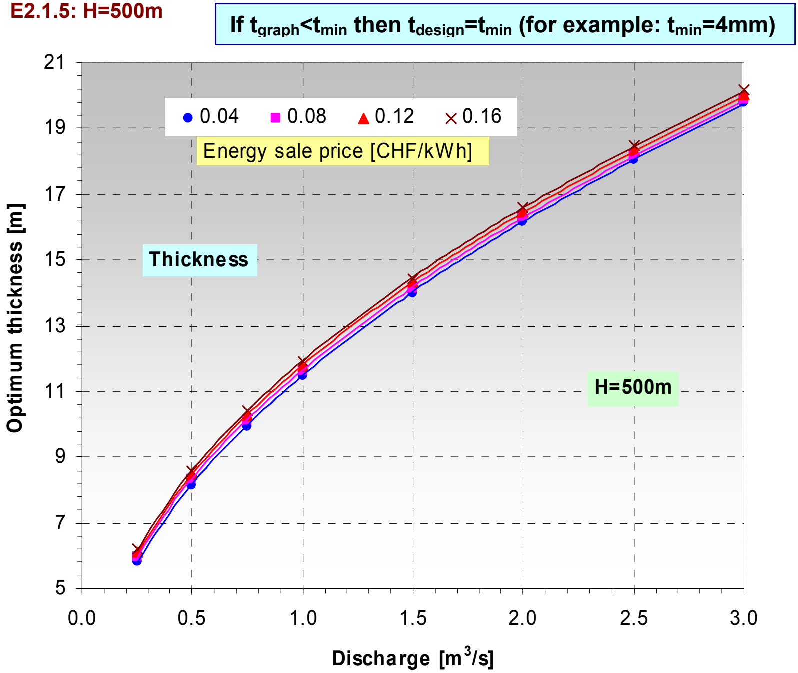


0.04
0.08
0.12
0.16
Energy sale price [CHF/kWh]
Diameter
H=500m
Optimum diameter [m]
Discharge [m3/s]
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.0
0.5
1.0
1.5
2.0
2.5
3.0

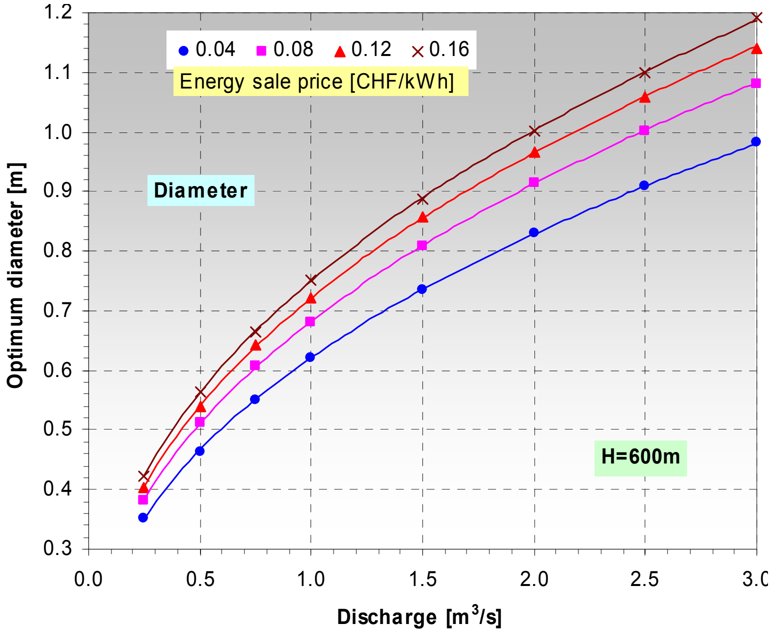


0.04
0.08
0.12
0.16
Energy sale price [CHF/kwh]
Velocity
H=500m
Optimum velocity [m/s]
Discharge [m3/s]
3.8
3.6
3.4
3.2
3.0
2.8
2.6
2.4
2.2
2.0
1.8
1.6
0.0
0.5
1.0
1.5
2.0
2.5
3.0

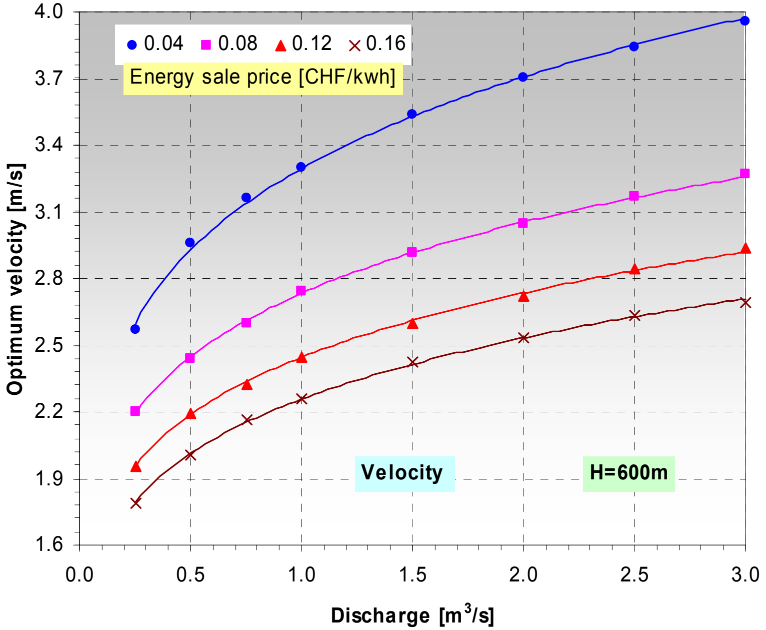
E2.1.5: H=500m



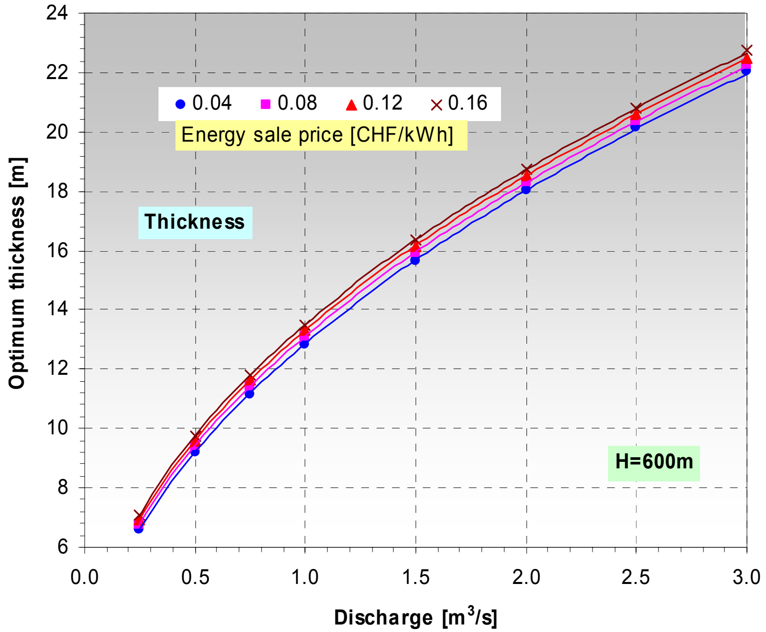
E2.1.6: H=600m



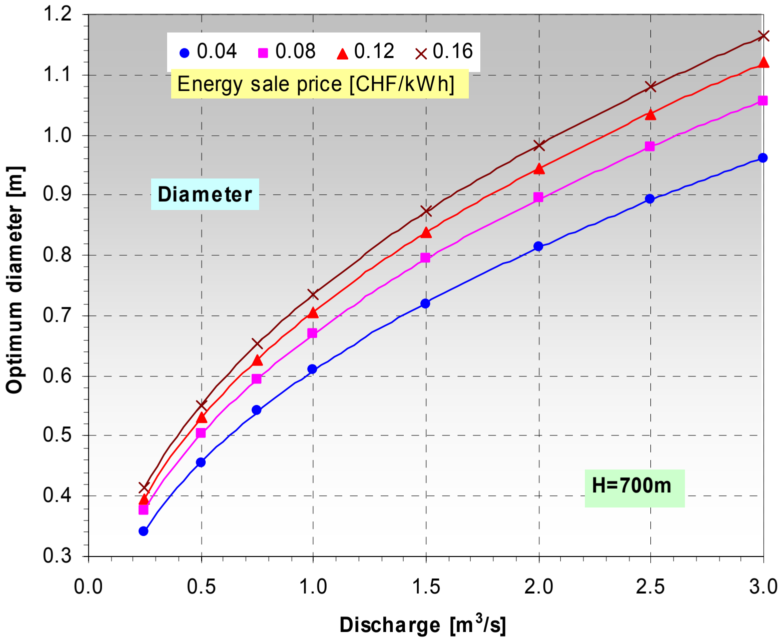
E2.1.6: H=600m



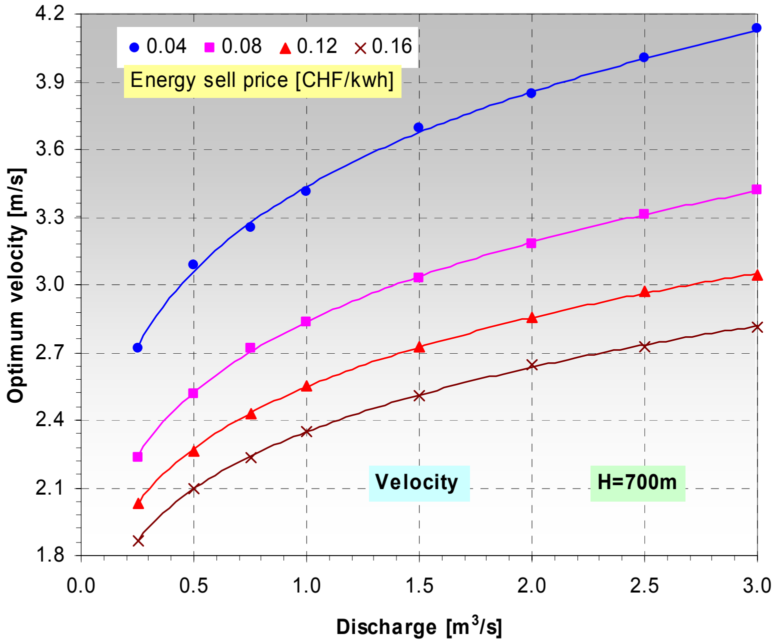
If $t_{graph}<t_{min}$ then $t_{design}=t_{min}$ (for example: t_{min}=4mm)



E2.1.7: H=700m

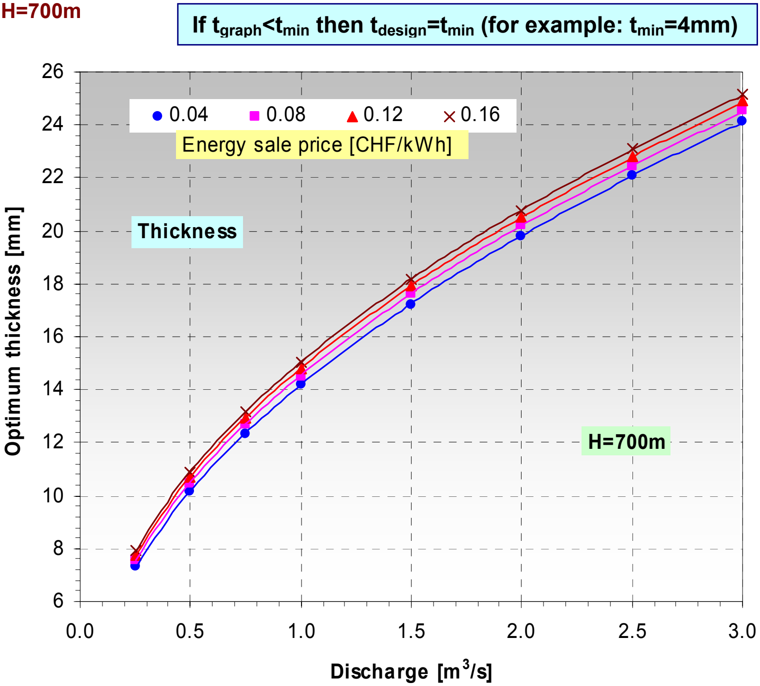


0.04
0.08
0.12
0.16
Energy sale price [CHF/kWh]
Diameter
H=700m
Optimum diameter [m]
Discharge [m3/s]
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.0
0.5
1.0
1.5
2.0
2.5
3.0

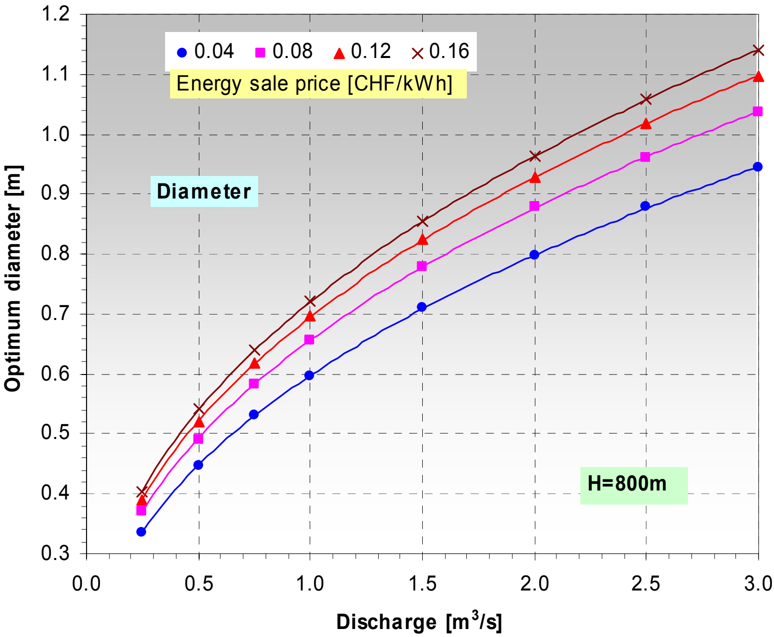


0.04
0.08
0.12
0.16
Energy sell price [CHF/kwh]
Velocity
H=700m
Optimum velocity [m/s]
Discharge [m3/s]
4.2
3.9
3.6
3.3
3.0
2.7
2.4
2.1
1.8
0.0
0.5
1.0
1.5
2.0
2.5
3.0

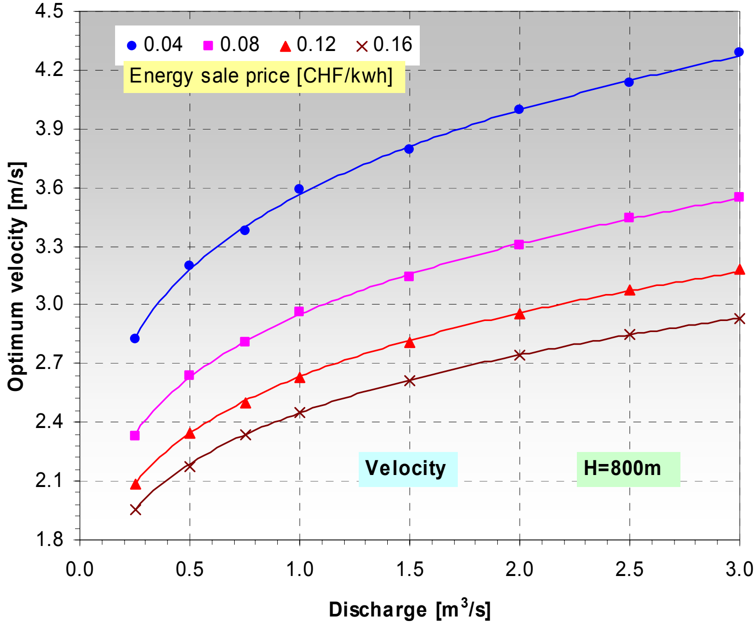
E2.1.7: H=700m



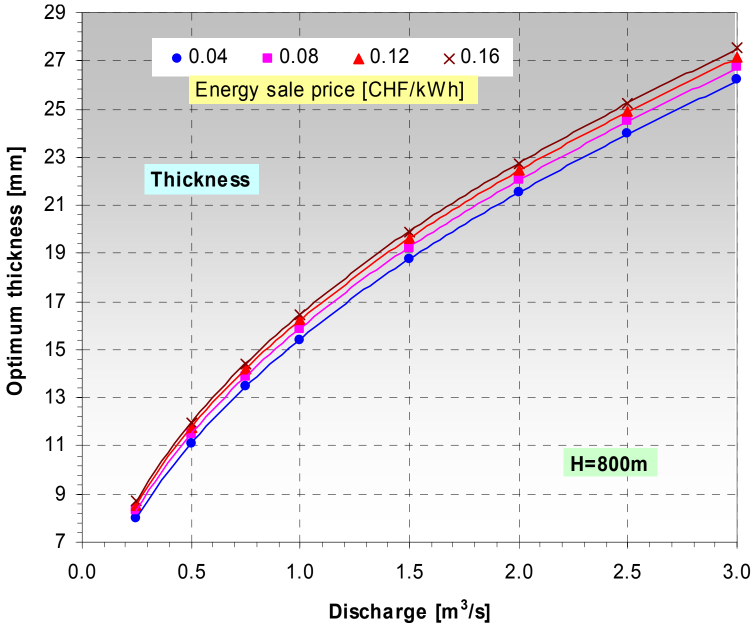
E2.1.8: H=800m



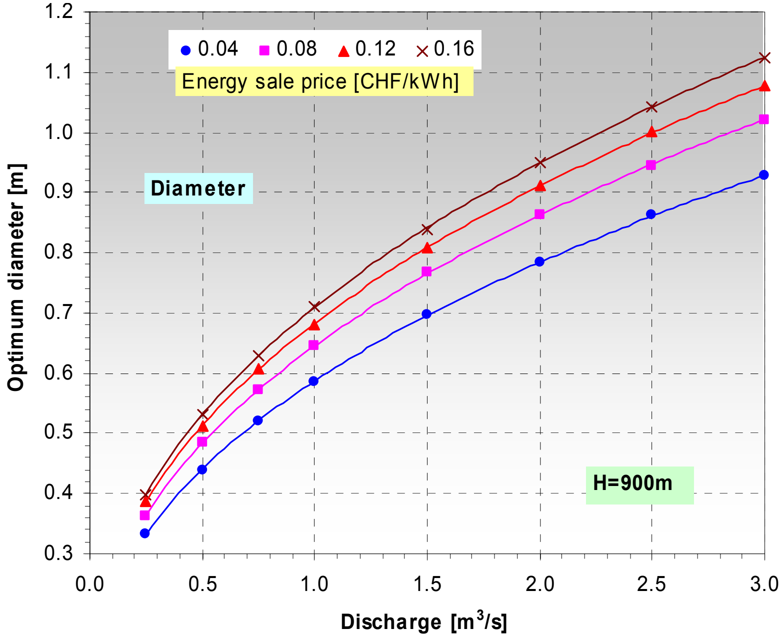
E2.1.8: H=800m

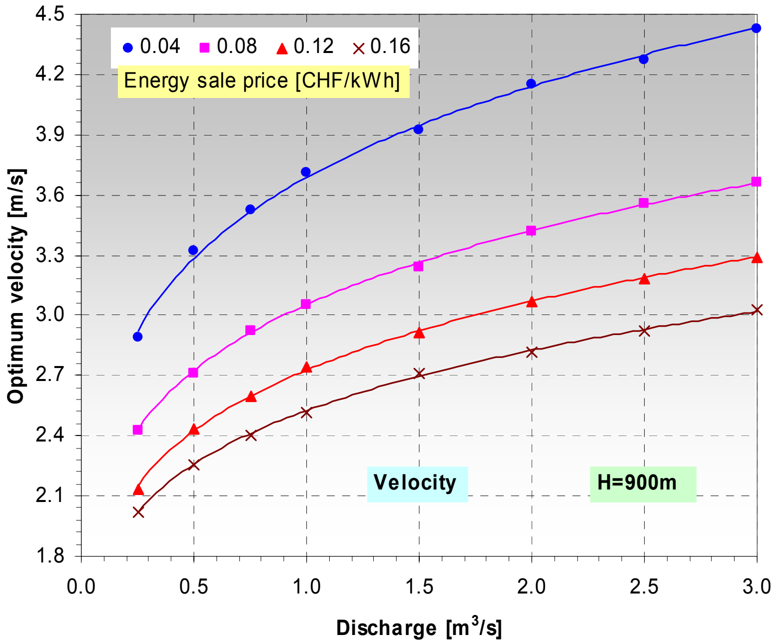


If $t_{graph}<t_{min}$ then $t_{design}=t_{min}$ (for example: t_{min}=4mm)

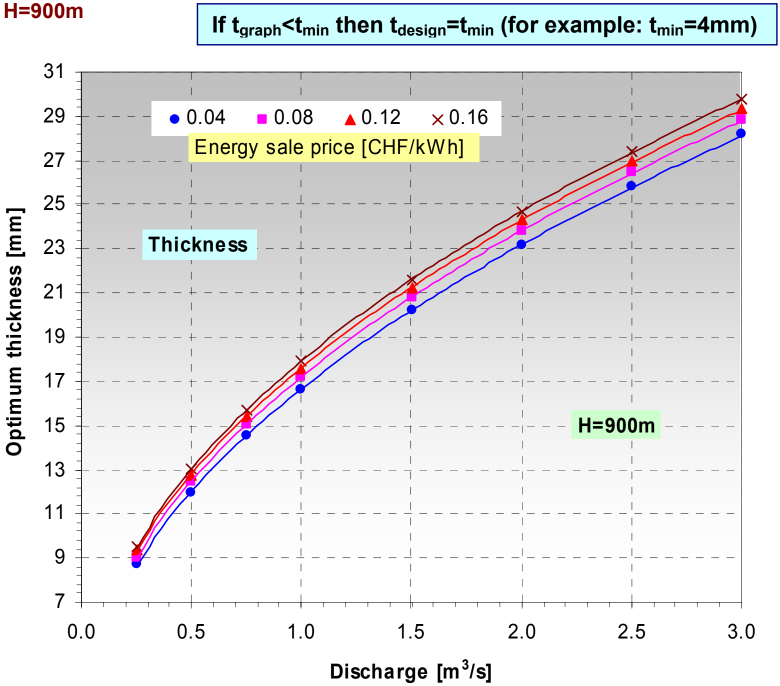


E2.1.9: H=900m

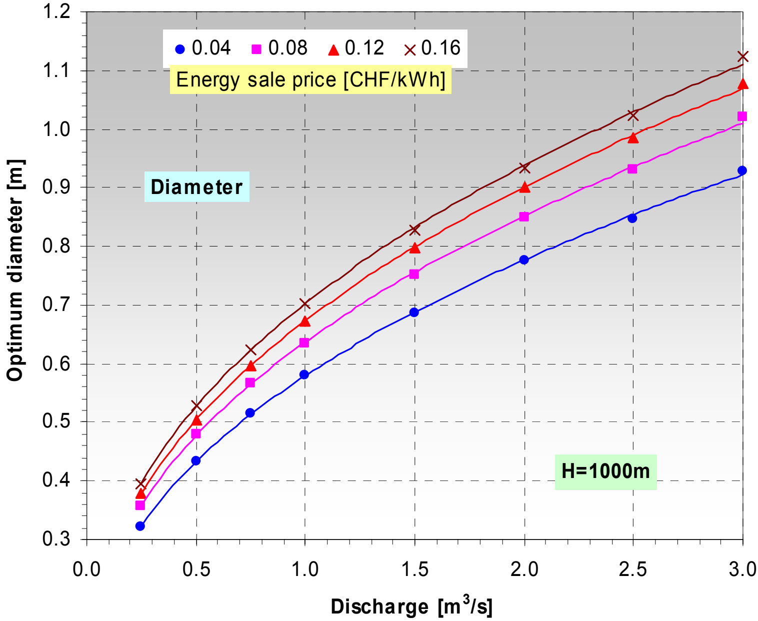

0.04
0.08
0.12
0.16
Energy sale price [CHF/kWh]
Diameter
H=900m
Optimum diameter [m]
Discharge [m³/s]
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.0
0.5
1.0
1.5
2.0
2.5
3.0


0.04
0.08
0.12
0.16
Energy sale price [CHF/kWh]
Velocity
H=900m
Optimum velocity [m/s]
Discharge [m³/s]
4.5
4.2
3.9
3.6
3.3
3.0
2.7
2.4
2.1
1.8
0.0
0.5
1.0
1.5
2.0
2.5
3.0

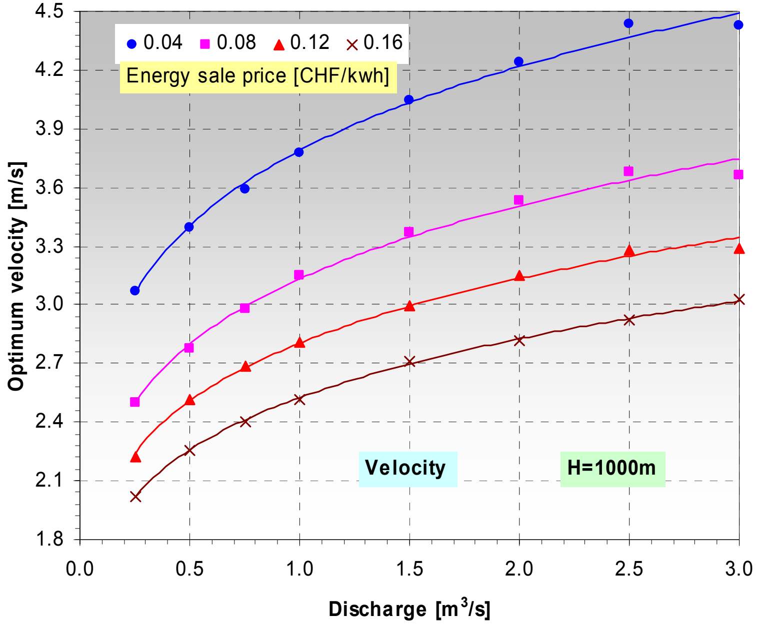
E2.1.9: H=900m



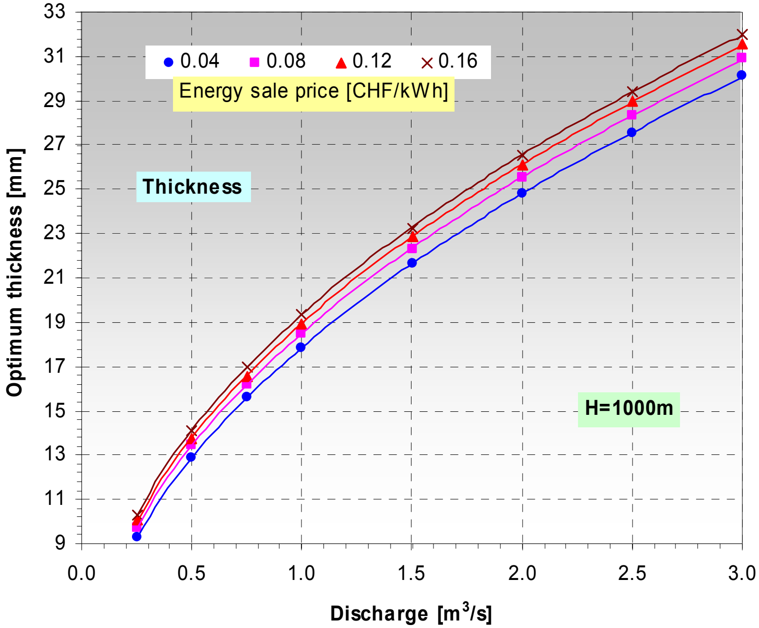
E2.1.10: H=1000m



E2.1.10: H=1000m

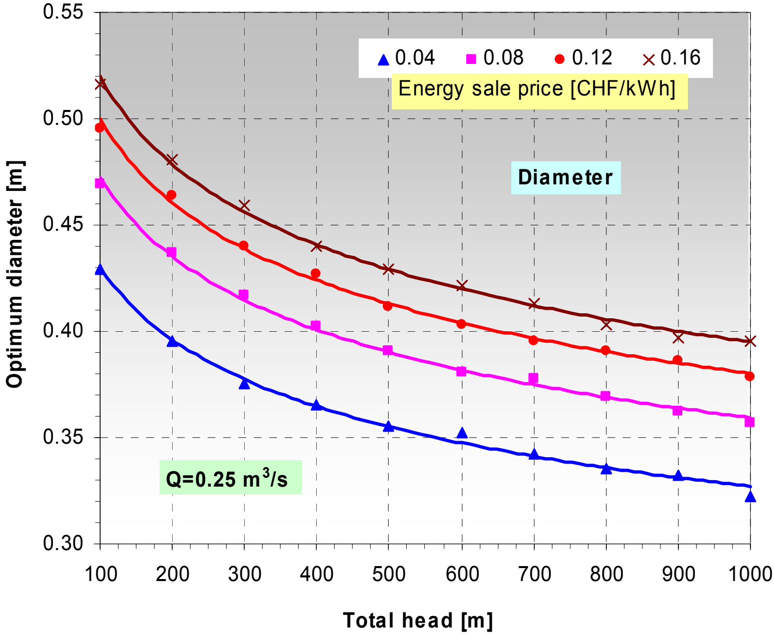


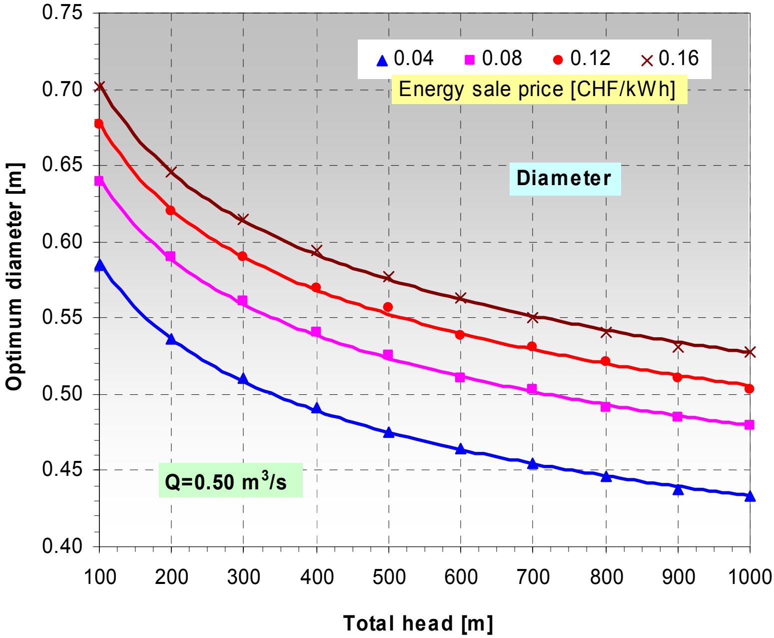
If $t_{graph}<t_{min}$ then $t_{design}=t_{min}$ (for example: t_{min}=4mm)



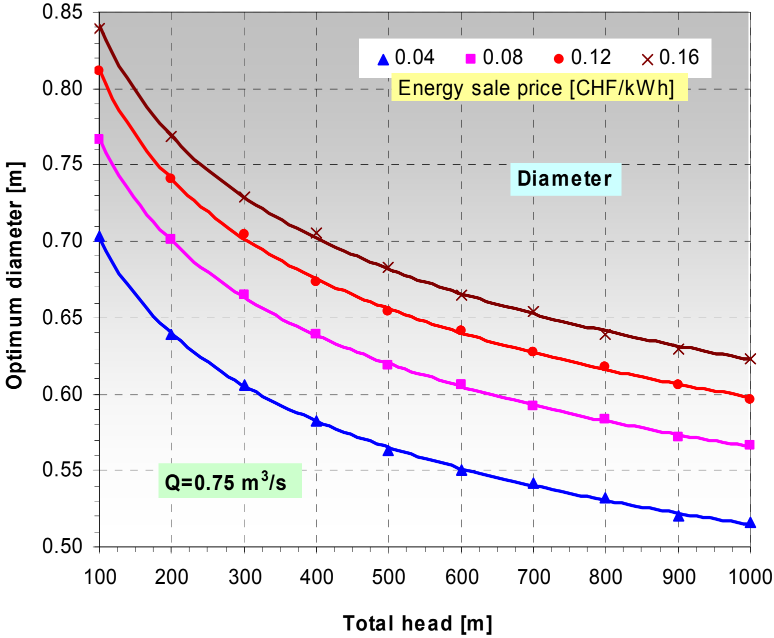
E2.2: Optimum penstock diameter and velocity as a function of total head and energy sale price for different discharges

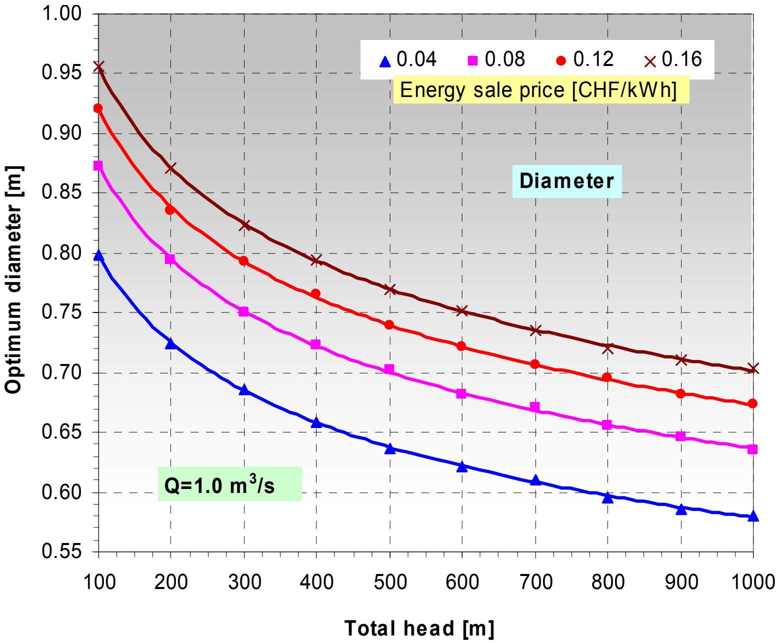
E2.2.1: Q=0.25 & 0.5m³/s



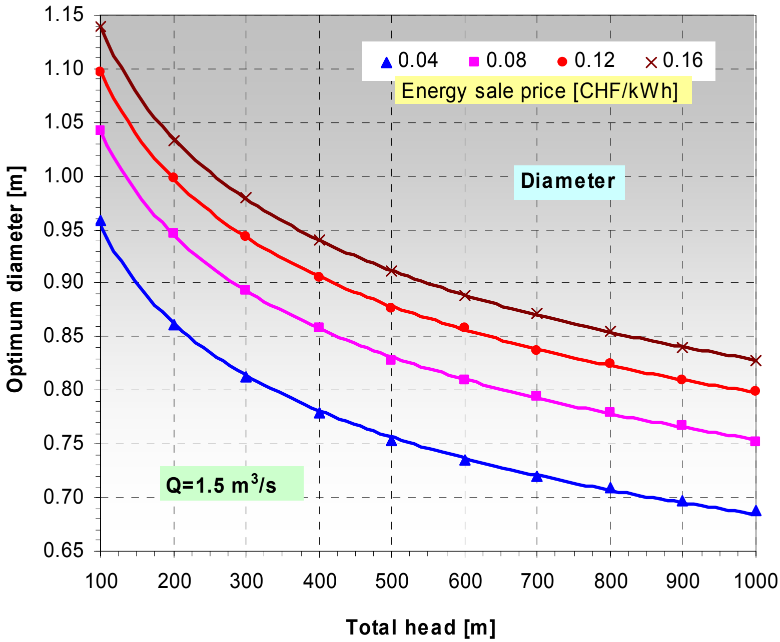


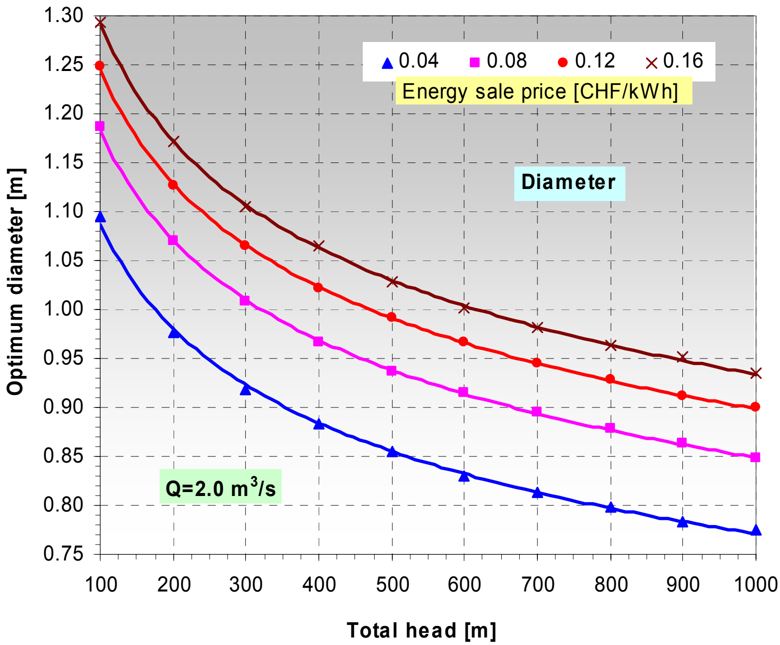
E2.2.2: Q=0.75 & 1.0m^3/s



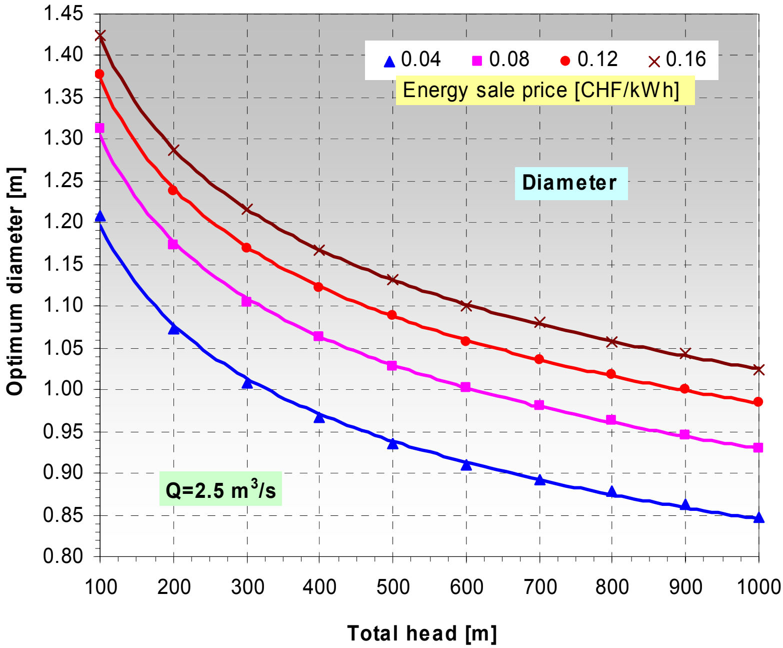


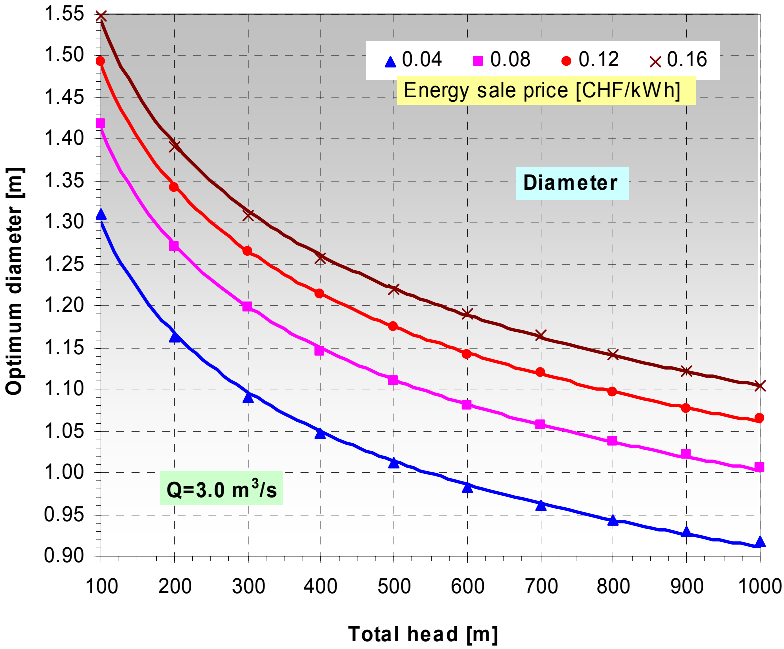
E2.2.3: Q=1.5 & 2.0m^3/s



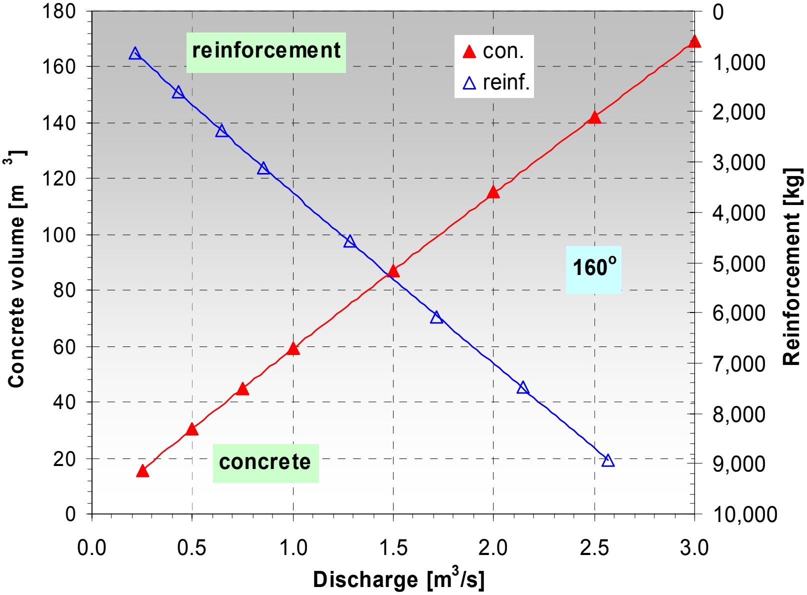


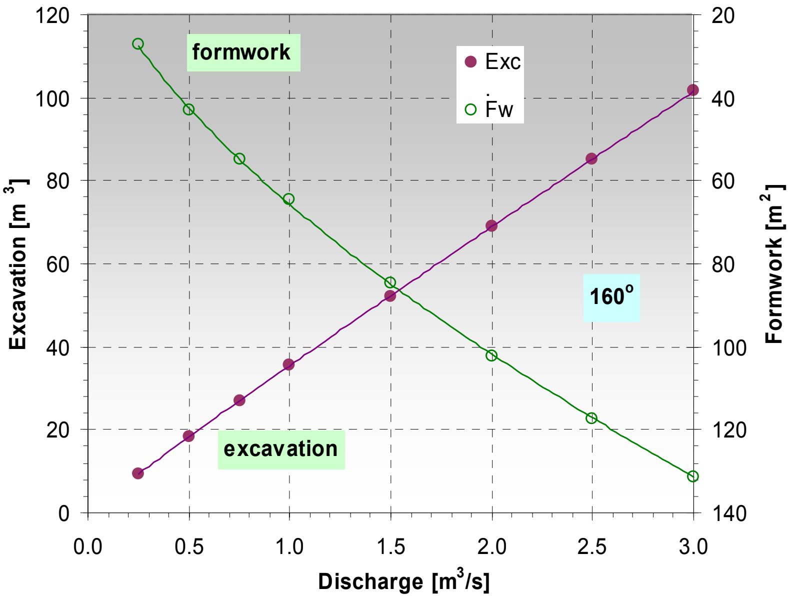
E2.2.4: Q=2.5 & 3.0m^3/s





E2.3: Concrete volume, reinforcement, excavation and formwork of anchor block as a function of discharge (angle=160°, H=100m, E.S.P. =0.16 CHF/kWh)





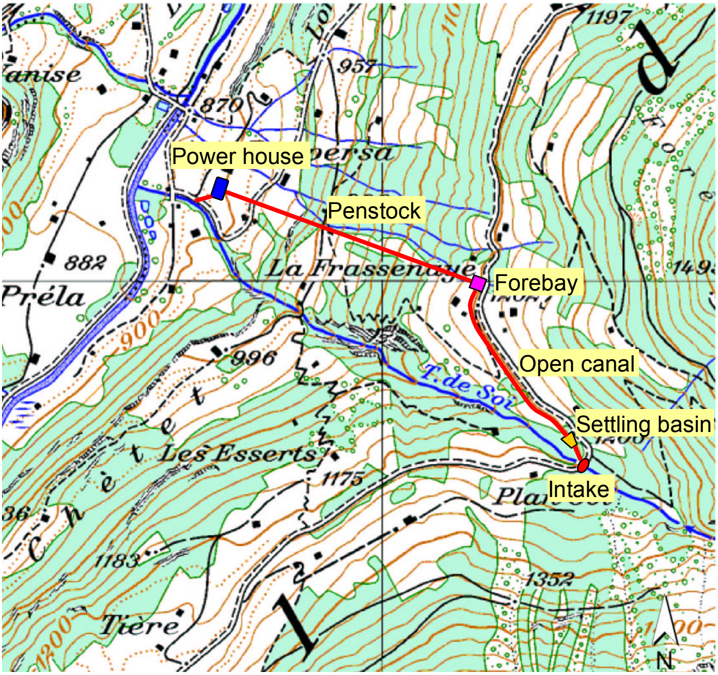
For other values: "POPEHYE Ver. 2.2"

Appendix F

Design example

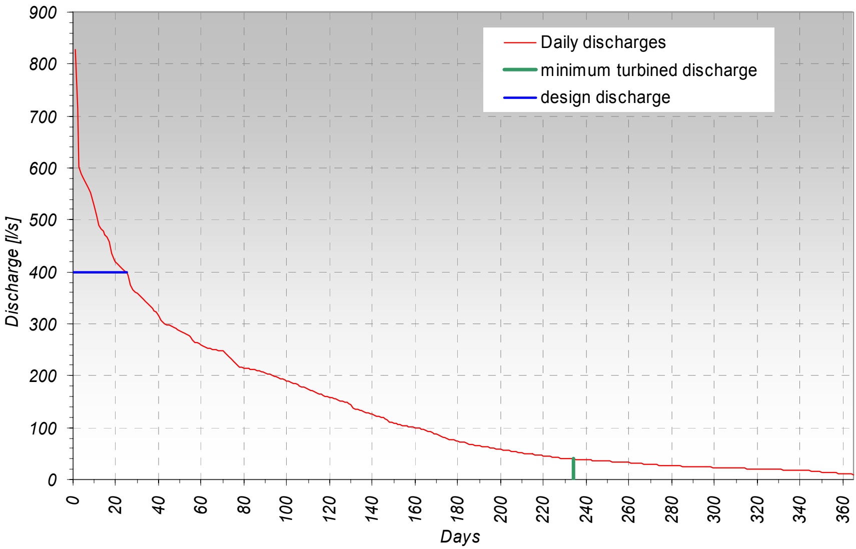
SHP project: Val d'Illiez in Switzerland

River: torrent de Soi



Main component of a SHP project for Soi (general plan)

F1: Flow duration curve



F2: Unit costs

Unit costs

Concrete volume [m^3]	Steel bar weight [kg]	Formwork surface [m^2]	Excavation in Rock [m^3]	Excavation in Alluvium [m^3]	Backfilling [m^3]	Project study [%]	Site installation [%]	Unit
250	2.5	50	100	30	50	10	30	CHF
CHF	CHF	CHF	CHF	CHF	CHF			

F3: Water intake

Tyrolian **B=1.1 m** **L=1.8 m**

Design discharge	River width	River bed type	Trashrack spaces	Slope angle of trashrack
0.40 [m^3/s]	W=5 [m]	Rock	a/b [1/2]	β [30°]

9. Beta=30, a/b=1/2, rock

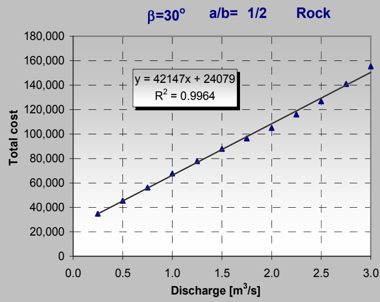
Q_{design}= 0.40 m^3/s
w= 5.0 m

9.1. Volume of civil works

Concrete volume m^3	Steel bar kg	Formwork m^2	Excavation m^3
32	**4953**	**87**	**41**

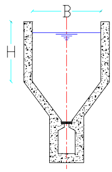
9.2. Cost of civil works

Concrete volume CHF	Steel bar CHF	Formwork CHF	Excavation CHF	Total cost CHF
7,986	**12,382**	**4,364**	**4,088**	**41,212**



Standard format for intake in optimization process and cost function (POPEHYE Ver2.2)

F4: Settling basin



Bieri **B=1.8 m** **H=1.4 m** **L=16 m**

Design discharge	Design grain size	Basin type	Bed type
0.40 [m^3/s]	d [0.3 mm]	Bieri	Rock

1. Bieri settling basin, d=0.3 mm, rock

Q_{design}= 0.40 m^3/s
L= 16 m

1.1. Volume of civil works

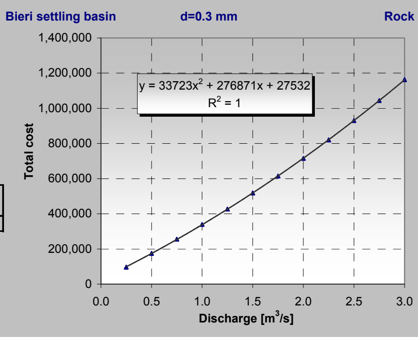
Concrete volume m^3	Steel bar kg	Formwork m^2	Excavation m^3	Backfilling m^3
49	**4718**	**332**	**269**	**121**

1.2. Cost of civil works

Concrete volume CHF	Steel bar CHF	Formwork CHF	Excavation CHF	Backfilling CHF	Total cost CHF
12,300	**11,795**	**16,585**	**26,869**	**6,070**	**105,276**

1.3. Cost of mechanical devices

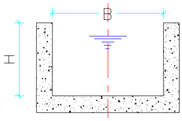
Unit cost CHF	Cost CHF
2,500	**40,000**



Standard format for settling basin in optimization process and cost function (POPEHYE Ver2.2)

F5: Headrace canal

Open canal **B=1.0 m** **H=0.6 m**



Design discharge	Bed type	Canal length	Profile	Construction material	Flow regime	Canal slope
0.40 [m^3/s]	Rock	450 [m]	Open canal	concrete	Free surface	0.2%

1.Open canal (concrete) , S=0.001 to 0.004, rock

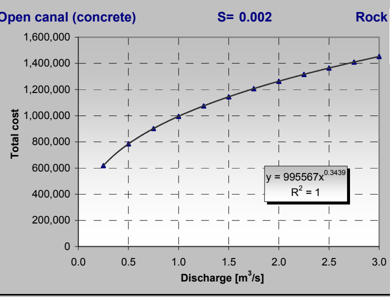
Q_{design}= 0.40 m^3/s
S= 2.0 ‰
L= 450 m

1.1. Volume of civil works

Concrete volume m^3	Steel bar kg	Formwork m^2	Excavation m^3	Backfilling m^3
297	**28706**	**1263**	**2214**	**1550**

1.2. Cost of civil works

Concrete volume CHF	Steel bar CHF	Formwork CHF	Excavation CHF	Backfilling CHF	Total cost CHF
74,207	**71,766**	**63,173**	**221,414**	**77,495**	**726,518**



Standard format for headrace canal in optimization process and cost function (POPEHYE Ver2.2)

F6: Forebay

Forebay **A=67 m²** **H=3.6 m**

Design discharge	bed type
0.40 [m^3/s]	Rock

1.Forebay with weir, rock

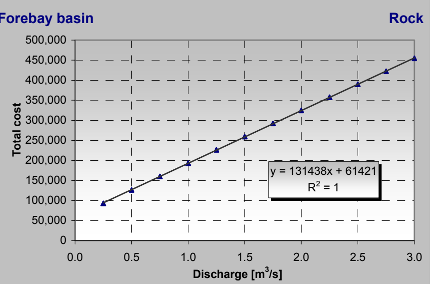
Q_{design}= 0.40 m^3/s

1.1. Volume of civil works

Concrete volume m^3	Steel bar kg	Formwork m^2	Excavation m^3
44	**5186**	**186**	**458**

1.2. Cost of civil works

Concrete volume CHF	Steel bar CHF	Formwork CHF	Excavation CHF	Total cost CHF
11,101	**12,964**	**9,285**	**45,799**	**113,185**



Standard format for forebay in optimization process and cost function (POPEHYE Ver2.2)

F7: Penstock

Penstock

D=0.55 m

t=6 mm

Design discharge	energy sale price	Head	Penstock length
0.40 [m^3/s]	0.126[CHF/kWh]	300 [m]	600 [m]

1. Penstock

H= 300 m
Q= 0.40 m^3/s
L= 600 m
E.S.P. 16 cts/kWh

1.1. Penstock optimum dimensions

dopt= 0.56 m
topt= 5.4 mm

1.2. Penstock cost

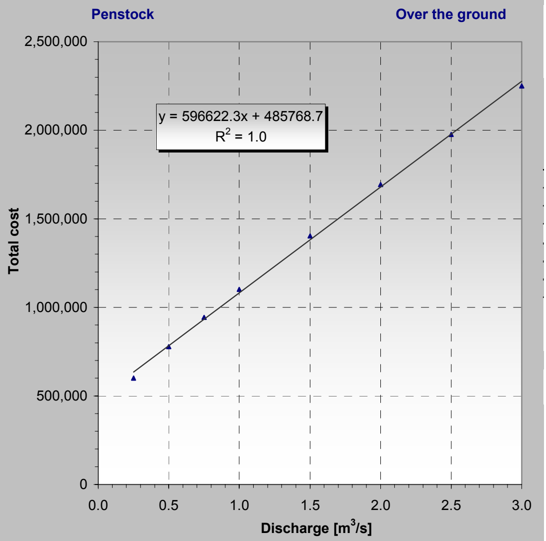
ρ 7850 kg/m^3
Steel cost= **272,076** CHF

Slope= 58 %
Ref. slope= 10 %
(s<=r.s.) a= 300
(s<=r.s.) b= 230
C=(a+b*d)*L
(s>r.s.) a= 600
(s>r.s.) b= 230

C=(a+b*d)*L

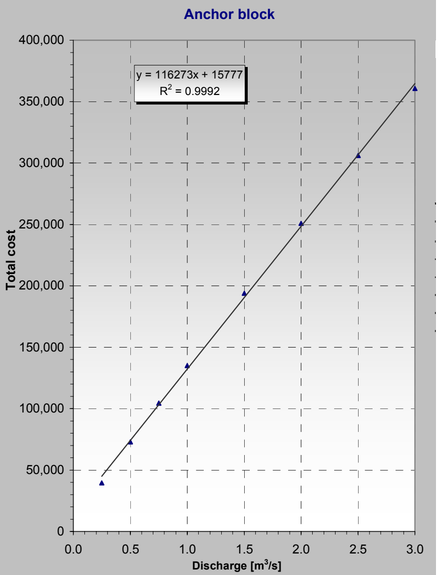
Installation cost= **437,206** CHF

Total cost= **709,281** CHF



2. Anchor blocks, alpha=120,140,160 ; e.s.p.=0.16

α= 140 m



2.1. Volume of civil works

Concrete volume	Steel bar	Formwork	Excavation
m^3	kg	m^2	m^3
85	**4466**	**89**	**51**

2.2. Cost of civil works

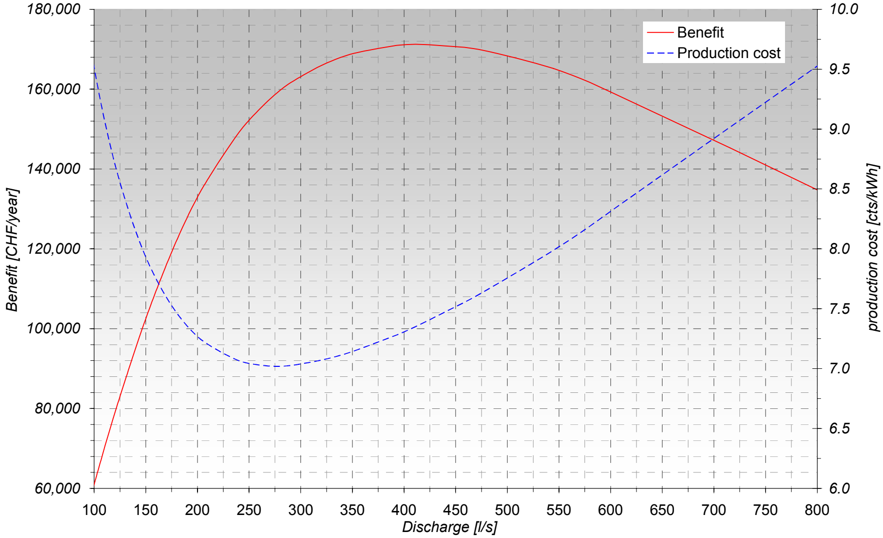
Concrete volume	Steel bar	Formwork	Excavation
CHF	CHF	CHF	CHF
21,167	**11,166**	**4,438**	**5,080**

Total cost
CHF
59,847

Standard format for Penstock optimization process and cost function (POPEHYE Ver2.2)

F8: Final results

E.S.P.=16 cts/kWh



Net benefit and production cost as a function of discharge (POPEHYE Ver2.2)

Q_{design} =400 lit/s

F9: Total cost

Components	Cost [CHF]	%
Intake	41,200	2
Settling basin	143,800	6
Headrace canal	727,000	31
Forebay	114,000	5
Penstock	710,000	31
Anchor block	60,000	3
Acces road	20,000	1
Power house	511,000	22
Total cost	**2,327,000**	100

Total cost for Q_{design}=0.4 m^3/s (POPEHYE Ver2.2)

P=830 kW

E=1970000 kWh

Appendix G

Drawings and specifications

NOTES:

1– ALL MEASUREMENTS AND DIMENSIONS SHOULD BE CONTROLLED BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

2– ALL CONSTRUCTION TASKS SHOULD COMPLY WITH THE TECHNICAL SPECIFICATIONS OR OTHERWISE AS STATED BY THE ENGINEER.

3– FOR OTHER VALUES OF ''BETA'' AND RATIO ''a/b'' REFER TO THE TECHNICAL REPORT AND PROGRAM POPEHYE (VER. 2.2)

4– THE ORIGINAL GROUND LINE IS SHOWN SCHEMATICALLY AND SHOULD BE ADOPTED WITH THE TOPOGRAPHIC CONDITION OF THE SITE.

LEGEND:

O.G.L –ORIGINAL GROUND LINE IN ROCK

Q –DESIGN DISCHARGE [m3/s]

B –WIDTH OF TRASHRACK

a,b –OPENINGS OF TRASHRACK [mm]

BETA –INCLINATION ANGLE OF TRASHRACK [°]

					3
					2
					1
SIGN.	DATE	SIGN.	DATE	DESCRIPTION:	REV.
APPROVED		CHECKED			

	DEPT. CODE:	CLIENT: THEMATIC NETWORK ON SMALL HYDROPOWER ENGINEERING WORK GROUP
	PROJ. CODE:	
NAME & SIGNATURE	DATE	ECOLE POLYTECHNIQUE FEDERALE DE LAUSANNE (EPFL) LABORATORY OF HYDRAULIC CONSTRUCTIONS (LCH)
M. ANDAROODI		DESIGNED
M. ANDAROODI		DRAWN
A. SCHLEISS		CHECKED
J.L. BOILLAT		TECHNICAL SEC. (APPROVED)
E. BOLLAERT		PROJ. (APPROVED)
A. SCHLEISS		DEPT. (APPROVED)
HYDRAULICS		FIELD

DWG. TITLE

STANDARDIZATION AND DESIGN RULES FOR CIVIL ENGINEERING WORKS

WATER INTAKE

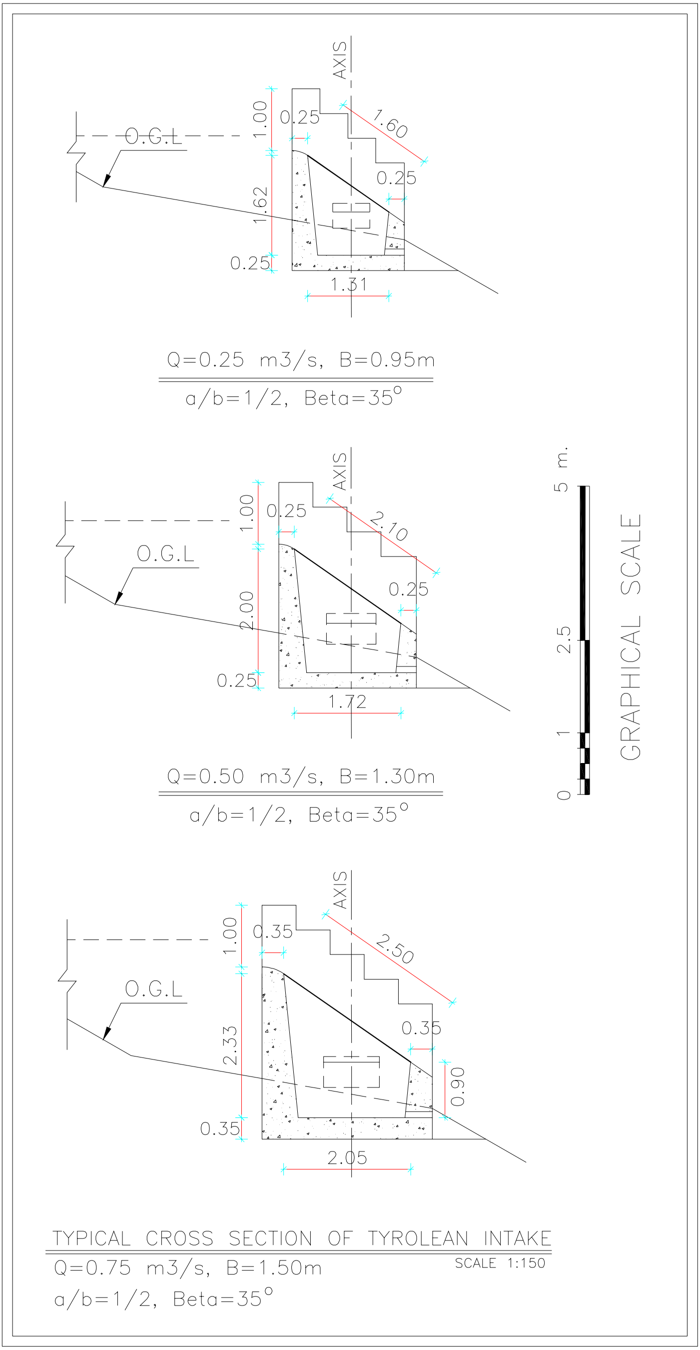
TYROLEAN INTAK , a/b=0.5 , BETA=35°

SCALE:

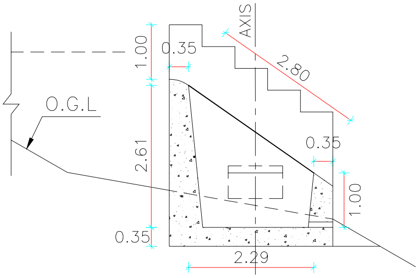
m.

UNIT:

DWG. No. HD–IN–TY–1

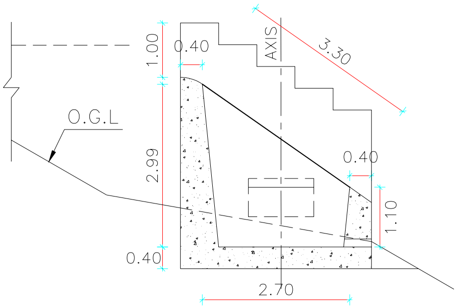


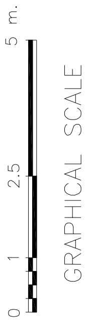
AXIS
O.G.L
1.00
0.25
1.60
0.25
1.62
0.25
1.31
Q=0.25 m3/s, B=0.95m
a/b=1/2, Beta=35°
AXIS
O.G.L
1.00
0.25
2.10
0.25
2.00
0.25
1.72
Q=0.50 m3/s, B=1.30m
a/b=1/2, Beta=35°
5 m.
2.5
1
0
GRAPHICAL SCALE
AXIS
O.G.L
1.00
0.35
2.50
0.35
2.33
0.90
0.35
2.05
TYPICAL CROSS SECTION OF TYROLEAN INTAKE
Q=0.75 m3/s, B=1.50m
SCALE 1:150
a/b=1/2, Beta=35°



Q=1.00 m3/s, B=1.70m

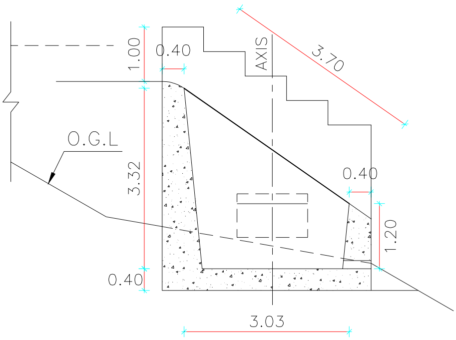
a/b=1/2, Beta=35°





Q=1.50 m3/s, B=2.00m

a/b=1/2, Beta=35°

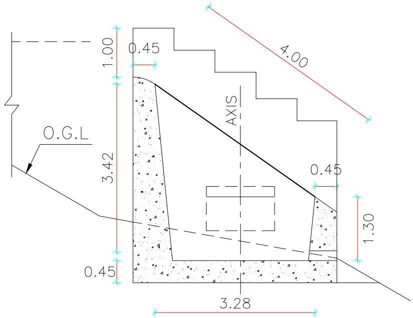


TYPICAL CROSS SECTION OF TYROLEAN INTAKE

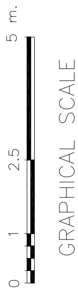
SCALE 1:150

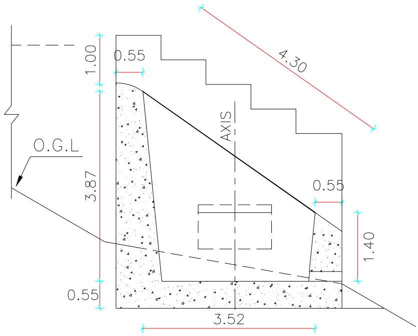
Q=2.00 m3/s, B=2.20m

a/b=1/2, Beta=35°

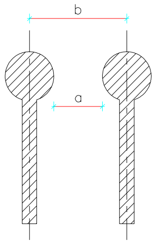


Q=2.50 m3/s, B=2.40m
a/b=1/2, Beta=35°





TYPICAL CROSS SECTION OF TYROLEAN INTAKE
SCALE 1:150
Q=3.00 m3/s, B=2.60m
a/b=1/2, Beta=35°



TYPICAL RACK SECTION

NOTES:

1– ALL MEASUREMENTS AND DIMENSIONS SHOULD BE CONTROLLED BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

2– ALL CONSTRUCTION TASKS SHOULD COMPLY WITH THE TECHNICAL SPECIFICATIONS OR OTHERWISE AS STATED BY THE ENGINEER.

3– FOR OTHER VALUE OF DESIGN GRAIN SIZE OF SEDIMENTS IN SETTLING BASIN REFER TO THE TECHNICAL REPORT AND PROGRAM POPEHYE (VER. 2.2)

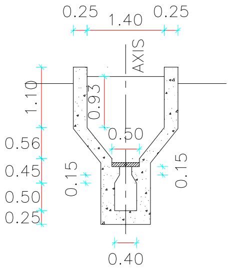
LEGEND:

Q –DESIGN DISCHARGE [m3/s]
d –SEDIMENT DESIGN GRAIN SIZE [mm]
L –SETTLING BASIN LENGTH [m]

					3
					2
					1
SIGN.	DATE	SIGN.	DATE	DESCRIPTION:	REV.
APPROVED		CHECKED			

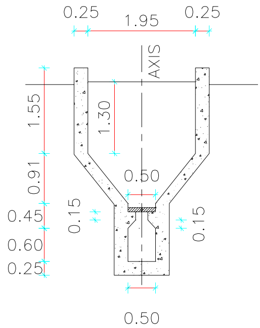
		DEPT. CODE:	CLIENT: THEMATIC NETWORK ON SMALL HYDROPOWER ENGINEERING WORK GROUP
		PROJ. CODE:	
NAME & SIGNATURE	DATE		ECOLE POLYTECHNIQUE FEDERALE DE LAUSANNE (EPFL) LABORATORY OF HYDRAULIC CONSTRUCTIONS (LCH)
M. ANDAROODI		DESIGNED	
M. ANDAROODI		DRAWN	
A. SCHLEISS		CHECKED	DWG. TITLE: STANDARDIZATION AND DESIGN RULES FOR CIVIL ENGINEERING WORKS SETTLING BASIN BIERI FLUSHING SYSTEM , d=0.3 mm
J.L. BOILLAT		APPROVED: TECHNICAL SEC.	
E. BOLLAERT		APPROVED: PROJ.	SCALE:
A. SCHLEISS		APPROVED: DEPT.	m.
HYDRAULICS		FIELD	DWG. No. HD–SB–BI–1 — UNIT:

TYPICAL CROSS SECTION OF SETTLING BASIN



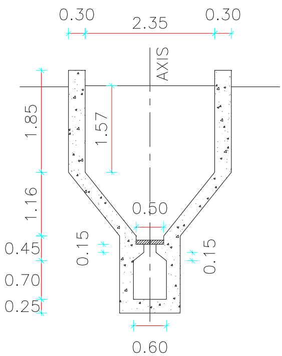
Q=0.25 m3/s , d=0.3mm

BIERI, L=12m



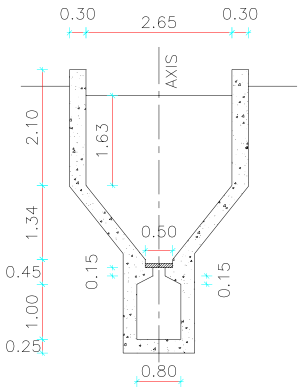
Q=0.50 m3/s , d=0.3mm

BIERI, L=17m



Q=0.75 m3/s , d=0.3mm

BIERI, L=21m

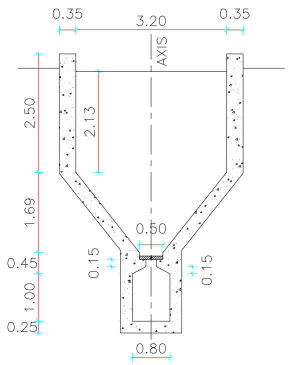


Q=1.00 m3/s , d=0.3mm

BIERI, L=23m

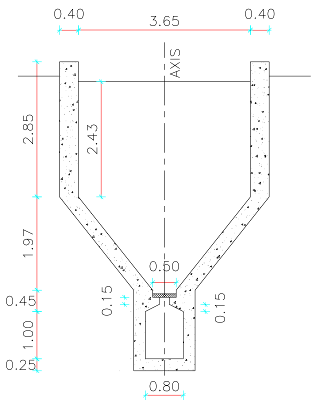
GRAPHICAL SCALE

TYPICAL CROSS SECTION OF SETTLING BASIN



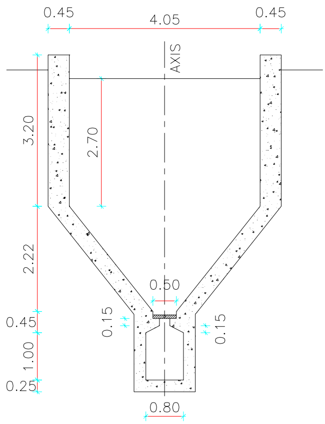
Q=1.50 m3/s , d=0.3mm

BIERI, L=28m



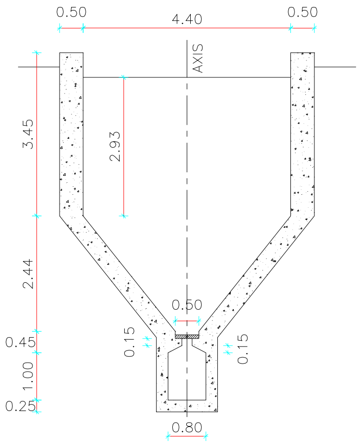
Q=2.00 m3/s , d=0.3mm

BIERI, L=32m



Q=2.50 m3/s , d=0.3mm

BIERI, L=36m



Q=3.00 m3/s , d=0.3mm

BIERI, L=39m

0 1 2.5 5 m.

GRAPHICAL SCALE

NOTES:

1– ALL MEASUREMENTS AND DIMENSIONS SHOULD BE CONTROLLED BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

2– ALL CONSTRUCTION TASKS SHOULD COMPLY WITH THE TECHNICAL SPECIFICATIONS OR OTHERWISE AS STATED BY THE ENGINEER.

3– FOR OTHER VALUE OF DESIGN GRAIN SIZE OF SEDIMENTS IN SETTLING BASIN REFER TO THE TECHNICAL REPORT AND PROGRAM POPEHYE VER. 2.2

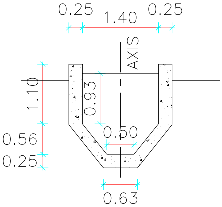
LEGEND:

Q –DESIGN DISCHARGE [m3/s]
d –SEDIMENT DESIGN GRAIN SIZE [mm]
L –SETTLING BASIN LENGTH [mm]

					3
					2
					1
SIGN.	DATE	SIGN.	DATE	DESCRIPTION:	REV.
APPROVED		CHECKED			

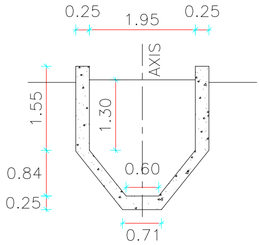
		DEPT. CODE:	CLIENT: THEMATIC NETWORK ON SMALL HYDROPOWER ENGINEERING WORK GROUP
		PROJ. CODE:	
NAME & SIGNATURE	DATE		ECOLE POLYTECHNIQUE FEDERALE DE LAUSANNE (EPFL) LABORATORY OF HYDRAULIC CONSTRUCTIONS (LCH)
M. ANDAROODI		DESIGNED	
M. ANDAROODI		DRAWN	DWG. TITLE
A. SCHLEISS		CHECKED	STANDARDIZATION AND DESIGN RULES FOR CIVIL ENGINEERING WORKS SETTLING BASIN BUCHI FLUSHING SYSTEM , d=0.3 mm
J.L. BOILLAT		APPROVED TECHNICAL SEC.	
E. BOLLAERT		APPROVED PROJ.	SCALE:
A. SCHLEISS		APPROVED DEPT.	m.
HYDRAULICS		FIELD	DWG. No. HD–SB–BU–1 UNIT:

TYPICAL CROSS SECTION OF SETTLING BASIN



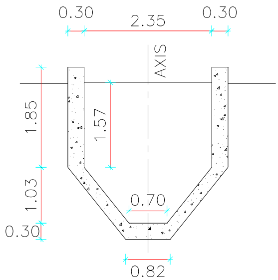
Q=0.25 m3/s , d=0.3mm

BUCHI, L=12m



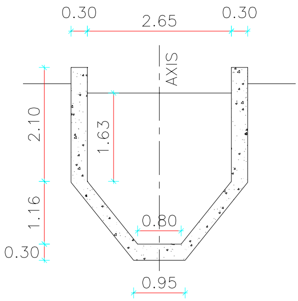
Q=0.50 m3/s , d=0.3mm

BUCHI, L=17m



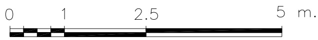
Q=0.75 m3/s , d=0.3mm

BUCHI, L=21m



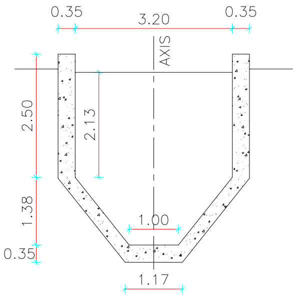
Q=1.00 m3/s , d=0.3mm

BUCHI, L=23m



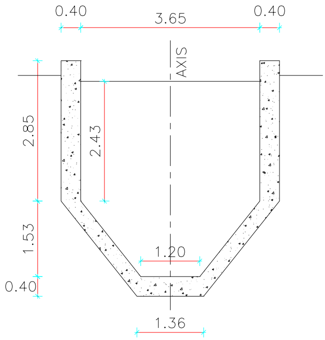
GRAPHICAL SCALE

TYPICAL CROSS SECTION OF SETTLING BASIN



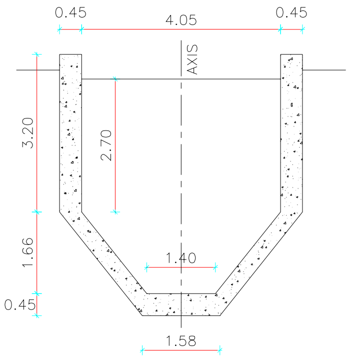
Q=1.50 m3/s , d=0.3mm

BUCHI, L=28m



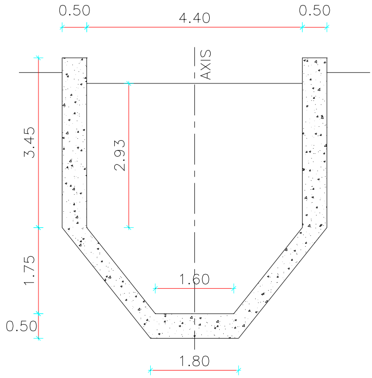
Q=2.00 m3/s , d=0.3mm

BUCHI, L=32m



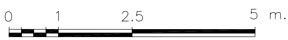
Q=2.50 m3/s , d=0.3mm

BUCHI, L=36m



Q=3.00 m3/s , d=0.3mm

BUCHI, L=39m



GRAPHICAL SCALE

NOTES:

1– ALL MEASUREMENTS AND DIMENSIONS SHOULD BE CONTROLLED BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

2– ALL CONSTRUCTION TASKS SHOULD COMPLY WITH THE TECHNICAL SPECIFICATIONS OR OTHERWISE AS STATED BY THE ENGINEER.

3– FOR OTHER VALUES OF HEADRACE SYSTEM SLOPE REFER TO THE TECHNICAL REPORT AND PROGRAM POPEHYE (VER. 2.2)

4– THE ORIGINAL GROUND LINE HAS IS SHOWN SCHEMATICALLY AND SHOULD BE ADOPTED WITH THE TOPOGRAPHIC CONDITION OF THE SITE.

LEGEND:

O.G.L –ORIGINAL GROUND LINE

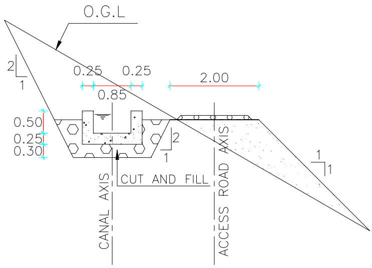
Q –DESIGN DISCHARGE [m3/s]

S –CANAL SLOPE

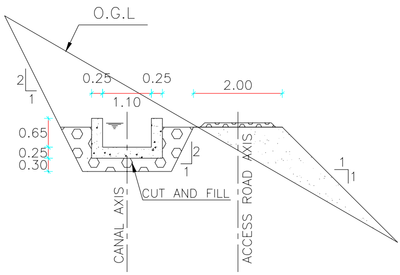
					3
					2
					1
SIGN.	DATE	SIGN.	DATE	DESCRIPTION:	REV.
APPROVED		CHECKED			

	DEPT. CODE:	CLIENT: THEMATIC NETWORK ON SMALL HYDROPOWER ENGINEERING WORK GROUP	
	PROJ. CODE:		
NAME & SIGNATURE	DATE		ECOLE POLYTECHNIQUE FEDERALE DE LAUSANNE (EPFL) LABORATORY OF HYDRAULIC CONSTRUCTIONS (LCH)
M. ANDAROODI		DESIGNED	
M. ANDAROODI		DRAWN	DWG. TITLE STANDARDIZATION AND DESIGN RULES FOR CIVIL ENGINEERING WORKS HEADRACE SYSTEM OPEN CONCRETE CANAL, S=0.001, FREE FLOW
A. SCHLEISS		CHECKED	
J.L. BOILLAT		APPROVED TECHNICAL SEC.	
E. BOLLAERT		APPROVED PROJ.	SCALE:
A. SCHLEISS		APPROVED DEPT.	m.
HYDRAULICS		FIELD	DWG. No. HD–HS–OC–1 UNIT:

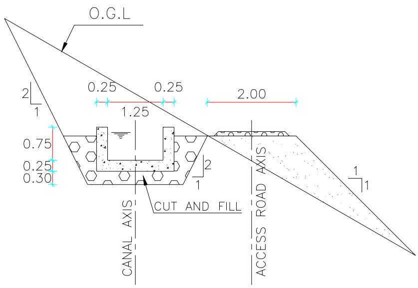
TYPICAL CROSS SECTION OF OPEN CANAL



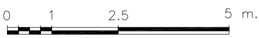
Q=0.25 m3/s , S=0.001



Q=0.50 m3/s , S=0.001

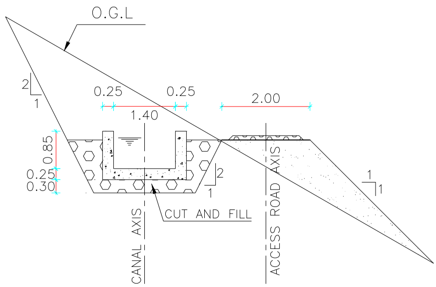


Q=0.75 m3/s , S=0.001

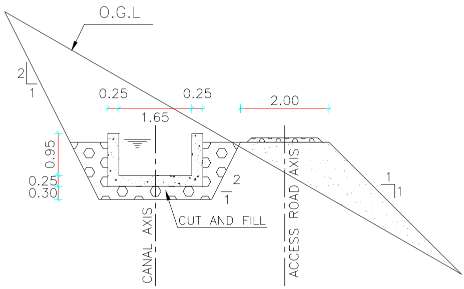


GRAPHICAL SCALE

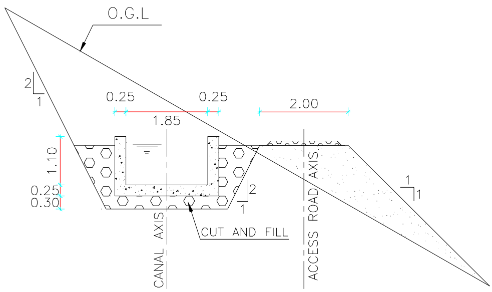
TYPICAL CROSS SECTION OF OPEN CANAL



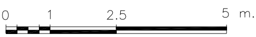
Q=1.00 m3/s , S=0.001



Q=1.50 m3/s , S=0.001

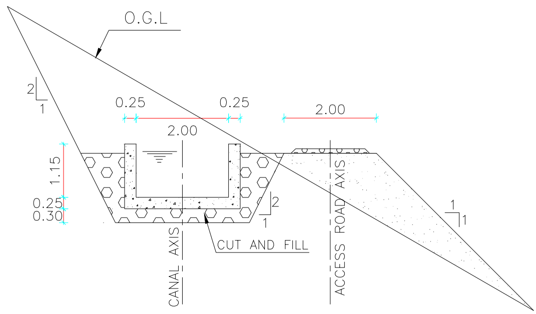


Q=2.00 m3/s , S=0.001

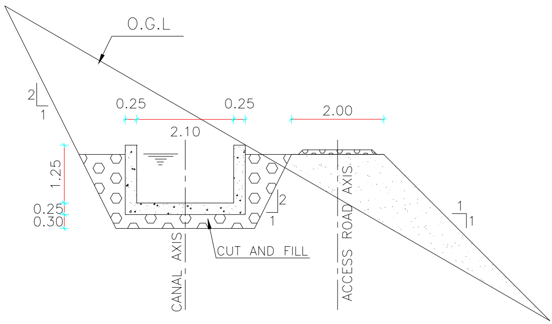


GRAPHICAL SCALE

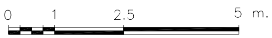
TYPICAL CROSS SECTION OF OPEN CANAL



Q=2.50 m3/s , S=0.001



Q=3.00 m3/s , S=0.001



GRAPHICAL SCALE

NOTES:

1– ALL MEASUREMENTS AND DIMENSIONS SHOULD BE CONTROLLED BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

2– ALL CONSTRUCTION TASKS SHOULD COMPLY WITH THE TECHNICAL SPECIFICATIONS OR OTHERWISE AS STATED BY THE ENGINEER.

3– FOR OTHER VALUES OF HEADRACE SYSTEM SLOPE REFER TO THE TECHNICAL REPORT AND PROGRAM POPEHYE (VER. 2.2)

4– THE ORIGINAL GROUND LINE IS SHOWN SCHEMATICALLY AND SHOULD BE ADOPTED WITH THE TOPOGRAPHIC CONDITION OF THE SITE.

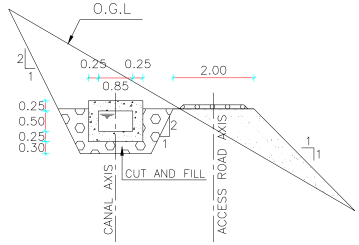
LEGEND:

O.G.L –ORIGINAL GROUND LINE
Q –DESIGN DISCHARGE [m3/s]
S –CANAL SLOPE

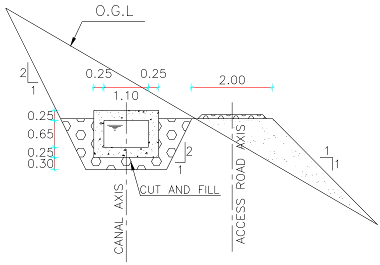
					3
					2
					1
SIGN.	DATE	SIGN.	DATE	DESCRIPTION:	REV.
APPROVED		CHECKED			

	DEPT. CODE:	CLIENT: THEMATIC NETWORK ON SMALL HYDROPOWER ENGINEERING WORK GROUP	
	PROJ. CODE:		
NAME & SIGNATURE	DATE		ECOLE POLYTECHNIQUE FEDERALE DE LAUSANNE (EPFL) LABORATORY OF HYDRAULIC CONSTRUCTIONS (LCH)
M. ANDAROODI		DESIGNED	
M. ANDAROODI		DRAWN	
A. SCHLEISS		CHECKED	DWG. TITLE STANDARDIZATION AND DESIGN RULES FOR CIVIL ENGINEERING WORKS HEADRACE SYSTEM BURIED CONCRETE CANAL, S=0.001, FREE FLOW
J.L. BOILLAT		APPROVED TECHNICAL SEC.	
E. BOLLAERT		APPROVED PROJ.	SCALE:
A. SCHLEISS		APPROVED DEPT.	m.
HYDRAULICS		FIELD	DWG. No. HD–HS–BC–2 UNIT:

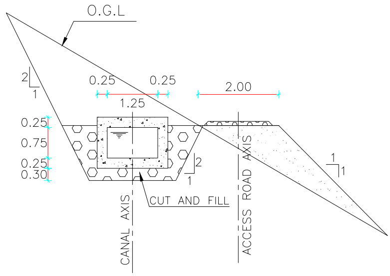
TYPICAL CROSS SECTION OF BURIED CANAL



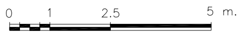
Q=0.25 m3/s , S=0.001



Q=0.50 m3/s , S=0.001

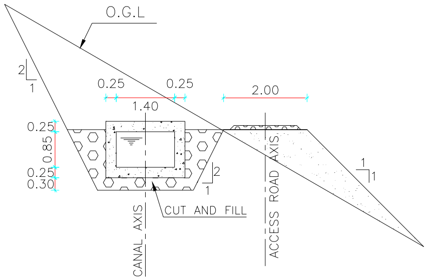


Q=0.75 m3/s , S=0.001

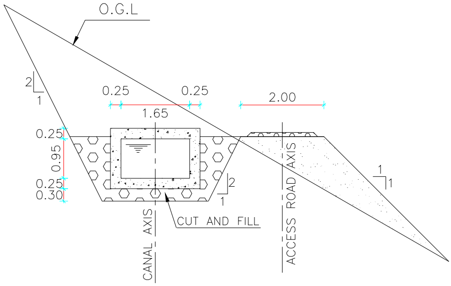


GRAPHICAL SCALE

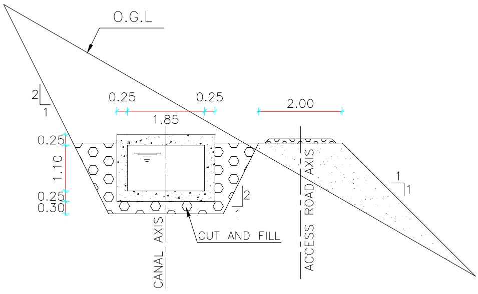
TYPICAL CROSS SECTION OF BURIED CANAL



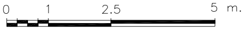
Q=1.00 m3/s , S=0.001



Q=1.50 m3/s , S=0.001

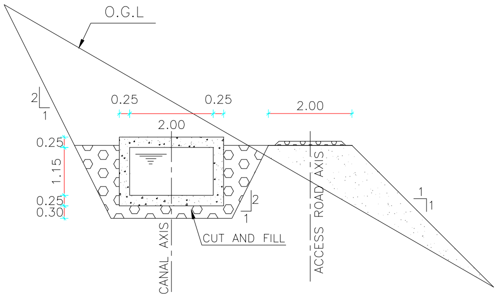


Q=2.00 m3/s , S=0.001

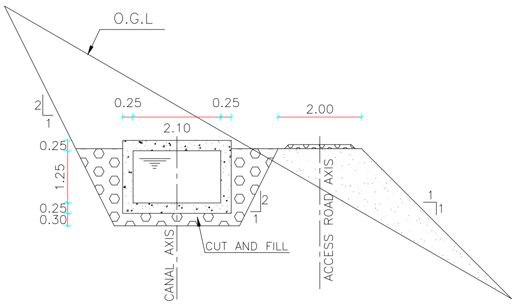


GRAPHICAL SCALE

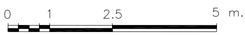
TYPICAL CROSS SECTION OF BURIED CANAL



Q=2.50 m3/s , S=0.001



Q=3.00 m3/s , S=0.001



GRAPHICAL SCALE

NOTES:

1– ALL MEASUREMENTS AND DIMENSIONS SHOULD BE CONTROLLED BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

2– ALL CONSTRUCTION TASKS SHOULD COMPLY WITH THE TECHNICAL SPECIFICATIONS OR OTHERWISE AS STATED BY THE ENGINEER.

3– FOR OTHER VALUES OF HEADRACE SYSTEM SLOPE REFER TO THE TECHNICAL REPORT AND PROGRAM POPEHYE (VER. 2.2)

4– THE ORIGINAL GROUND LINE IS SHOWN SCHEMATICALLY AND SHOULD BE ADOPTED WITH THE TOPOGRAPHIC CONDITION OF THE SITE.

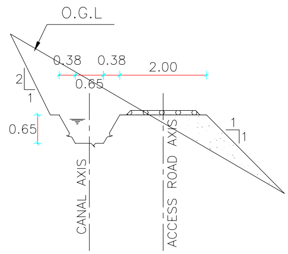
LEGEND:

O.G.L –ORIGINAL GROUND LINE
Q –DESIGN DISCHARGE [m3/s]
S –CANAL SLOPE

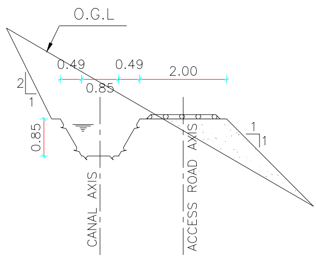
					3
					2
					1
SIGN.	DATE	SIGN.	DATE	DESCRIPTION:	REV.
APPROVED		CHECKED			

	DEPT. CODE:	CLIENT: THEMATIC NETWORK ON SMALL HYDROPOWER ENGINEERING WORK GROUP
	PROJ. CODE:	
NAME & SIGNATURE	DATE		ECOLE POLYTECHNIQUE FEDERALE DE LAUSANNE (EPFL) LABORATORY OF HYDRAULIC CONSTRUCTIONS (LCH)
M. ANDAROODI		DESIGNED	
M. ANDAROODI		DRAWN	
A. SCHLEISS		CHECKED	DWG. TITLE STANDARDIZATION AND DESIGN RULES FOR CIVIL ENGINEERING WORKS HEADRACE SYSTEM ROCKY CANAL , S=0.001 , FREE FLOW
J.L. BOILLAT		APPROVED TECHNICAL SEC.	
E. BOLLAERT		APPROVED PROJ.	SCALE:
A. SCHLEISS		APPROVED DEPT.	m.
HYDRAULICS		FIELD	DWG. No. HD–HS–RC–3	UNIT:

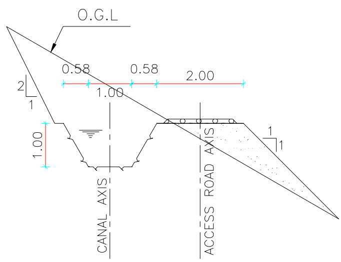
TYPICAL CROSS SECTION OF ROCKY CANAL



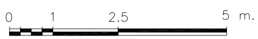
Q=0.25 m3/s , S=0.001



Q=0.50 m3/s , S=0.001

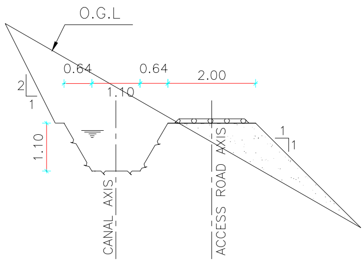


Q=0.75 m3/s , S=0.001

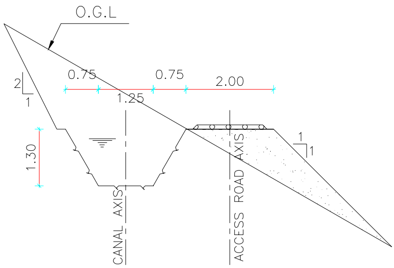


GRAPHICAL SCALE

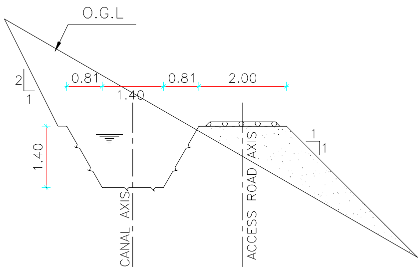
TYPICAL CROSS SECTION OF ROCKY CANAL



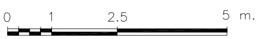
Q=1.00 m3/s , S=0.001



Q=1.50 m3/s , S=0.001

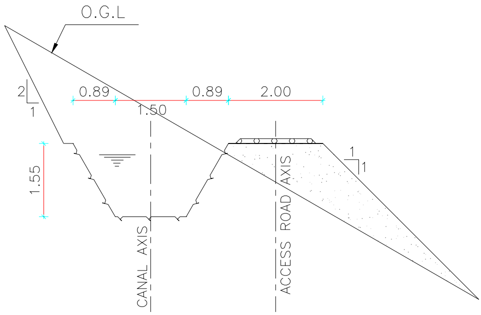


Q=2.00 m3/s , S=0.001

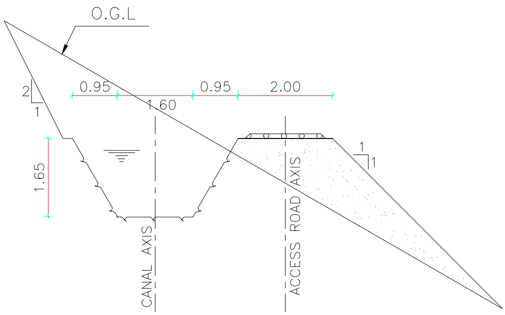


GRAPHICAL SCALE

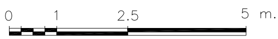
TYPICAL CROSS SECTION OF ROCKY CANAL



Q=2.50 m3/s , S=0.001



Q=3.00 m3/s , S=0.001



GRAPHICAL SCALE

NOTES:

1– ALL MEASUREMENTS AND DIMENSIONS SHOULD BE CONTROLLED BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

2– ALL CONSTRUCTION TASKS SHOULD COMPLY WITH THE TECHNICAL SPECIFICATIONS OR OTHERWISE AS STATED BY THE ENGINEER.

3– FOR OTHER VALUES OF HEADRACE SYSTEM SLOPE REFER TO THE TECHNICAL REPORT AND PROGRAM POPEHYE (VER. 2.2)

4– THE ORIGINAL GROUND LINE IS SHOWN SCHEMATICALLY AND SHOULD BE ADOPTED WITH THE TOPOGRAPHIC CONDITION OF THE SITE.

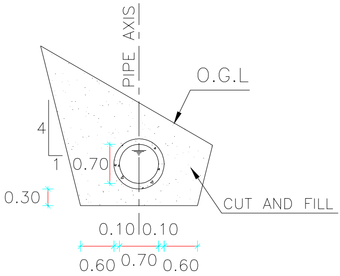
LEGEND:

O.G.L –ORIGINAL GROUND LINE
Q –DESIGN DISCHARGE [m3/s]
S –PIPE SLOPE

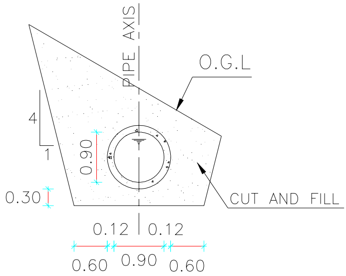
					3
					2
					1
SIGN.	DATE	SIGN.	DATE	DESCRIPTION:	REV.
APPROVED		CHECKED			

	DEPT. CODE:	CLIENT: THEMATIC NETWORK ON SMALL HYDROPOWER ENGINEERING WORK GROUP	
	PROJ. CODE:		
NAME & SIGNATURE	DATE		ECOLE POLYTECHNIQUE FEDERALE DE LAUSANNE (EPFL) LABORATORY OF HYDRAULIC CONSTRUCTIONS (LCH)
M. ANDAROODI		DESIGNED	
M. ANDAROODI		DRAWN	
A. SCHLEISS		CHECKED	DWG. TITLE STANDARDIZATION AND DESIGN RULES FOR CIVIL ENGINEERING WORKS
J.L. BOILLAT		APPROVED TECHNICAL SEC.	HEADRACE SYSTEM
E. BOLLAERT		APPROVED PROJ.	SCALE:
A. SCHLEISS		APPROVED DEPT.	BURIED CONCRETE PIPE, S=0.001, FREE FLOW
HYDRAULICS		FIELD	DWG. No. HD–HS–BP–4
			UNIT: m.

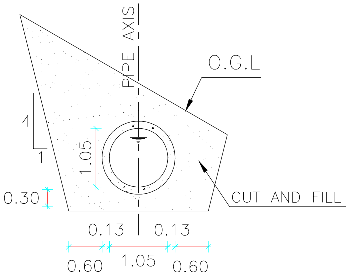
TYPICAL CROSS SECTION OF BURIED PIPE



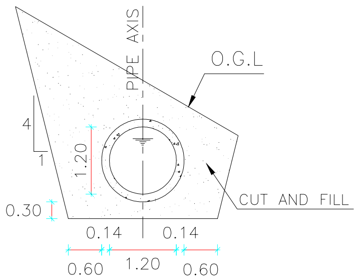
Q=0.25 m3/s , S=0.001



Q=0.50 m3/s , S=0.001



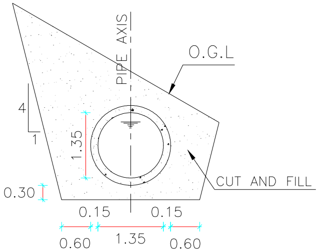
Q=0.75 m3/s , S=0.001



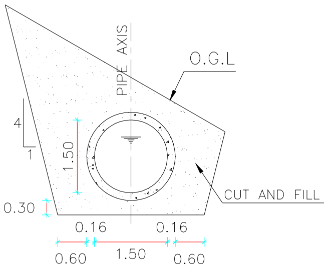
Q=1.00 m3/s , S=0.001

GRAPHICAL SCALE

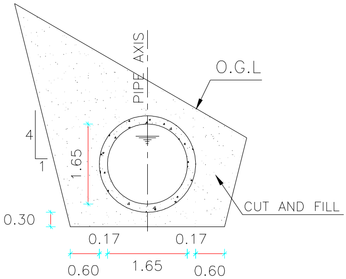
TYPICAL CROSS SECTION OF BURIED PIPE



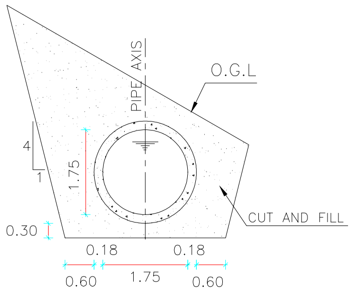
Q=1.50 m3/s , S=0.001



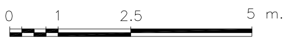
Q=2.00 m3/s , S=0.001



Q=2.50 m3/s , S=0.001



Q=3.00 m3/s , S=0.001



GRAPHICAL SCALE

Appendix H

Installation of POPEHYE

Program Installation

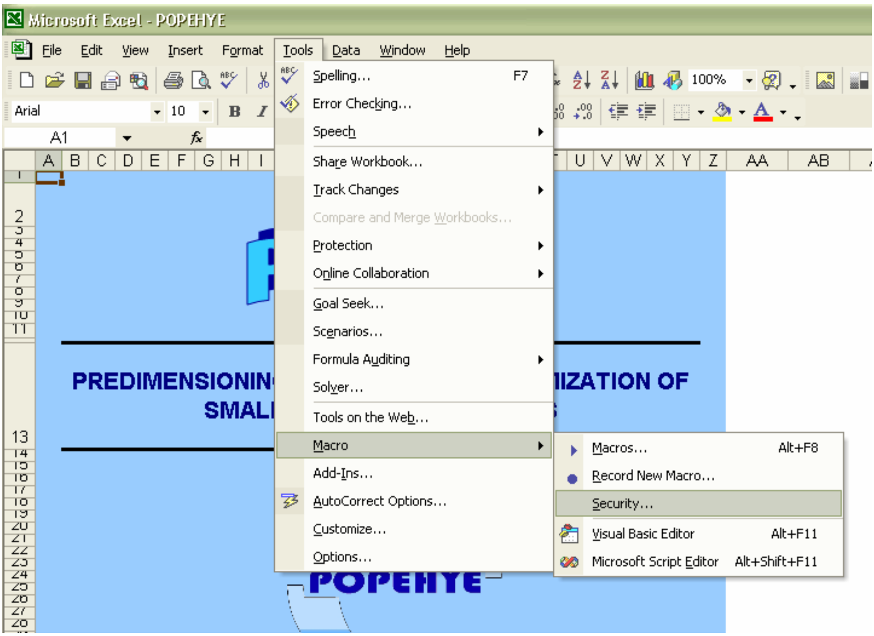
The program ''POPEHYE'' is upgraded under **Microsoft Excel 2002** in the domain of **Microsoft Windows XP 2002**.

This program is functioning under certain conditions and the user has to put in order the following stages:

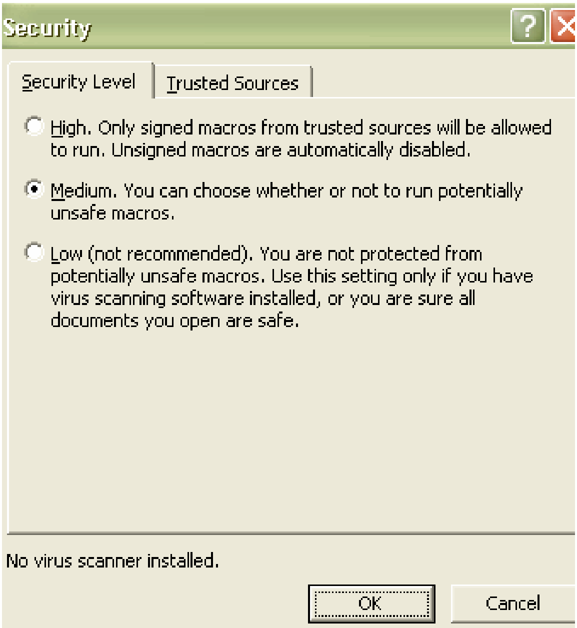
H1: Macro security

Before opening the program ''POPEHYE'' in Excel, the user should define the security level of Macro as below:

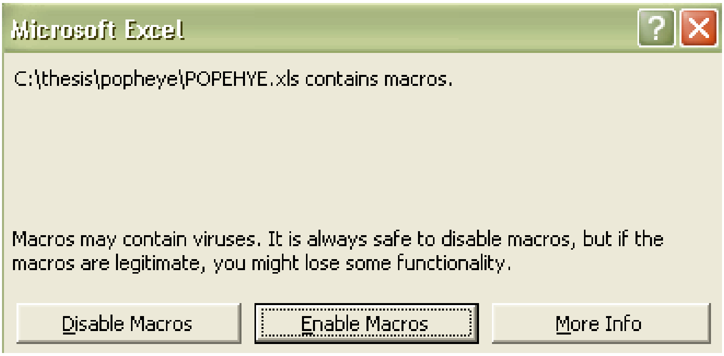
Excel menu ----> Tools -----> Macro -----> Security…



Then the Medium level of security for Macro should be chosen:



After this, when the user opens the program ‘‘POPEHYE’’, the following message will appear on the screen:

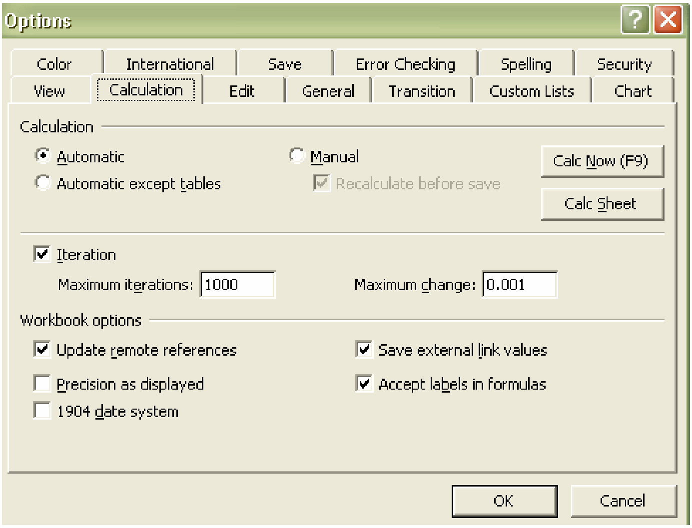


The user has to select ‘‘Enable Macros’’ in order to activate all of the internal written visual basic programs in ‘‘POPEHYE’’.

H2: Computation precision

Please introduce the following items in the calculation format of Excel:

Excel menu ----> Tools ----> Options… -----> Calculation

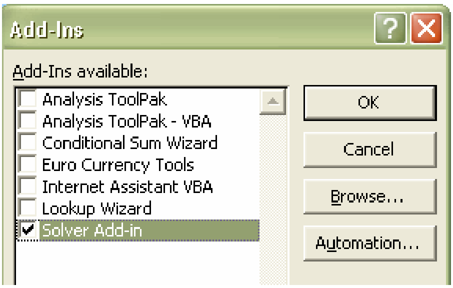


H3: Solver function

The "solver function" has to be added in two different phases. Please follow the process below:

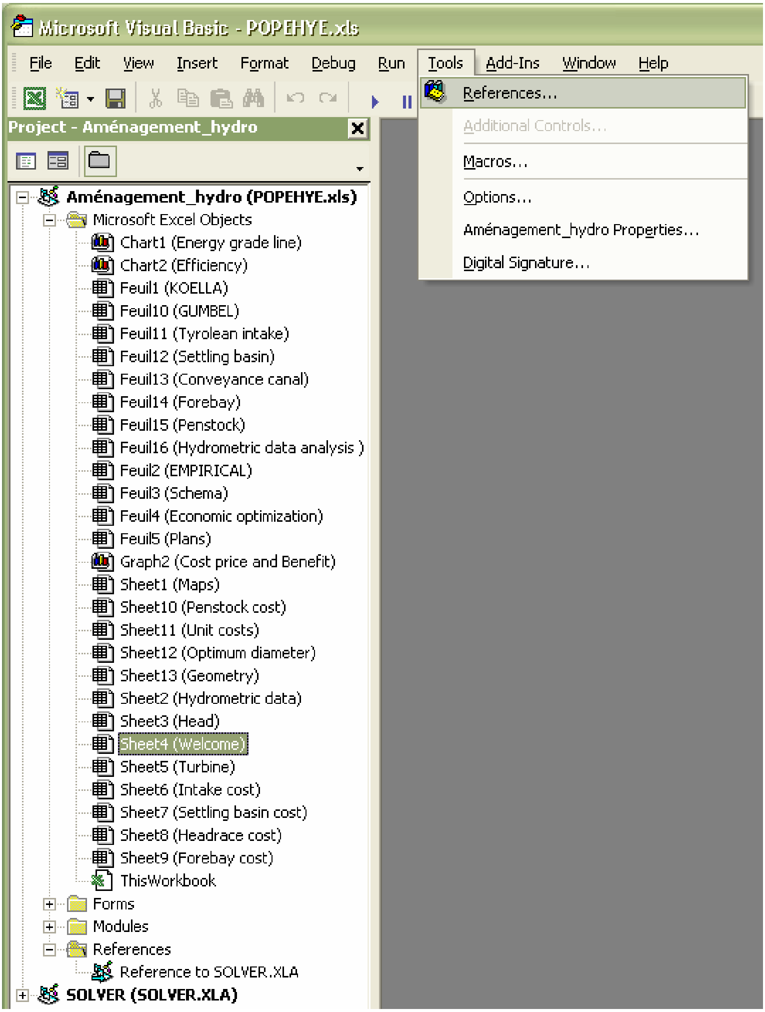
Excel menu ----> Tools -----> Add-Ins…

The following window will be appeared and the "solver function" has to be added by clicking from the list. After this, the program excel tries to install "solver" from the source CD or disk of Microsoft Office which should be available by the user.

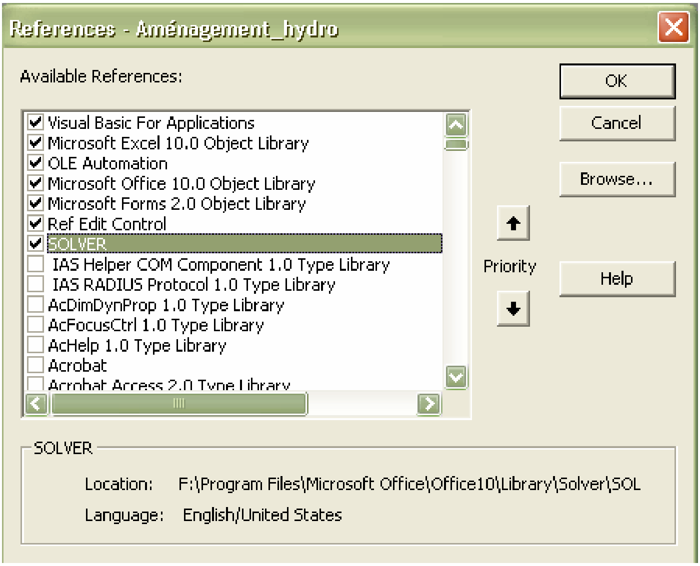


The second stage is to activate the "solver function" in the Microsoft Visual Basic window. The Visual basic menu is opened by pressing "Alt+F11" in the normal menu of Excel. Once the "VB window" is opened, the user has to install manually the "solver" as follows:

Microsoft Visual Basic menu ----> Tools -----> References…

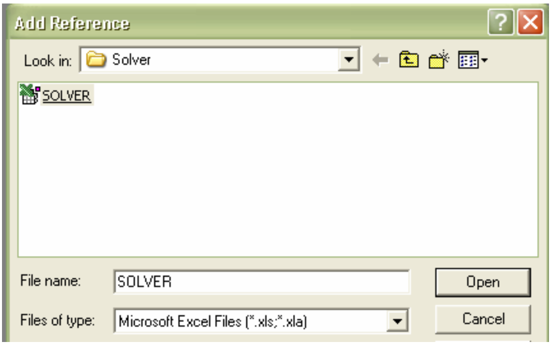


After clicking on references, the following window will be opened. Please select the "SOLVER" from the list below:



If the "SOLVER" does not exist in the above list, then please click on the "Browse…" and then find the file called "SOLVER.xla" which should be normally in the following address:

Hard disk drive\Program Files\Microsoft Office\Office10 or 11\Library\SOLVER.xla



At last please click on the "ok" and save the "POPEHYE.xls".

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